BRIDGE ENGINEERING

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BRIDGE ENGINEERING

BY

J. A. L. WADDELL

C.E. (Rens. Poly. Inst.); B.A.Sc., Ma.E., D.Sc. (McGill Univ.); D.E.
(Univ. of Neb.); LL.D. (Univ. of Mo.); Kogakuhakushi
(Doctor of Eng., Imp. Univ. of Japan)

Knight Commander of the Japanese Order of the Rising Sun; Membre de la Société de Bienfaisance de la Grande Duchesse Olga de Russie; Senior Partner of Waddell & Son, Consulting Engineers, Kansas City, Mo.; Member of the American Society of Civil Engineers; of the American Institute of Consulting Engineers; of the Franklin Institute; of the Institution of Civil Engineers, London; of La Société des Ingénieurs Civils, Paris; of the Canadian Society of Civil Engineers; of the Western Society of Engineers; of the Rensselaer Society of Engineers; of the Engineers' Club of Kansas City; of the Society for the Promotion of Engineering Education; of the American Association for the Advancement of Science; of the American Society for Testing Materials; of the International Society for Testing Materials; of the American Railway Engineering Association; of La Société de Géographie de France; of the Phi Beta Kappa Society; of the Tau Beta Pi Society; of the Sigma Xi Society; of the National Conservation Association; of the Economic League; and Honorary Member of the Kogaku Kyokai (Japanese Engineering Society), and of the Société Internationale d'Études de Correspondance et d'Échanges "Concordia," Paris

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TO

His Imperial Majesty, The Emperor of Japan

AS A MARK OF THE AUTHOR'S
DEEP APPRECIATION OF THE HIGH HONOR
CONFERRED UPON HIM BY THE

Japanese Government

ON THREE DISTINCT OCCASIONS, VIZ., IN 1882, 1888
AND 1915, AND AS AN EVIDENCE OF THE
PROFOUND ESTEEM AND REGARD IN WHICH THE AUTHOR
HOLDS THE ENTIRE JAPANESE NATION

THIS BOOK

THE GREATEST PROFESSIONAL WORK OF HIS LIFE
WITH HIS MAJESTY'S GRACIOUS PERMISSION
IS MOST RESPECTFULLY DEDICATED

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Dr. James H Trowell
2-1-1924

PREFACE

It is now more than eighteen years since the author's little work, *De Pontibus*, was given to the engineering profession, and nearly thirteen years since the second edition of it was issued. In the preface to the latter there were indicated ten new chapters for the third edition, the author then thinking that these would cover substantially everything additional that he would have to say concerning the subject of bridge engineering. How erroneous that impression was can be seen by noting the titles of the chapters of this book and the number of them, which, by the way, is three and a third times as great as the number of chapters in the first and second editions of *De Pontibus*, and two and a third times as great as that for the contemplated third edition. Moreover, the average length of the chapters in the new book is about twice as great as that in the old one, and the total amount of illustration is some thirty times as large, making the ratio of the volumes of the two contents fully seven to one.

Just after the first edition of *De Pontibus* was issued, the author began to prepare systematic analyses, digests, and records of all of his work, using diagrams whenever feasible; and he has continued that practice ever since, with the result that he has gradually accumulated a great fund of thoroughly digested and systematized information which has proved most valuable, in both office and field, to himself and his associates, and which, consequently, ought to be serviceable not only to the engineering profession in general but also to the higher officials of railroads and any others who may be interested in the building of bridges. It has long been a dream of the author to give this information to the profession; but he has recognized that to do so properly would involve an immense expenditure of both time and money. However, he believes that it is incumbent upon every member thereof to add his mite to the sum total of professional knowledge in order to repay in some slight measure the large obligation which the individual owes to his predecessors for the accumulated information handed down by them. Only by an altruistic, far-sighted policy of this kind can the profession be advanced to its position of greatest usefulness and thereby receive complete recognition of its value to society. An opposite policy would mean an arrested development of professional capacity, a gradual deterioration of engineering standards, and eventual stagnation.

For eight years after the first issue of *De Pontibus* the author was working under such high pressure that he could find no time for preparing the revision of that work as indicated in the preface to its second edition, then he began to despair of ever finding an opportunity before the inertia of advancing years would prevent the accomplishment of the task. Early in 1906, when the partnership of Waddell and Harrington was being arranged, one of the first subjects discussed was that of the writing of the contemplated third edition; and there was then outlined what it should contain. The list of chapters grew so rapidly that very soon it was decided to change the title to "Bridge Engineering," and ignore the idea of a third edition of the old book. It was intended that the preparation of the new book should be the joint work of the new firm, the author feeling then that the task would be too onerous for him to undertake single-handed. Unfortunately for the book, at least as far as its early completion was concerned, the firm developed an exceedingly large and important practice, amounting at one period, simultaneously in both office and field, to some fifteen million dollars' worth of bridge work, so that very little time could be spared for technical writing. From time to time, however, the author managed to prepare a few of the chapters allotted to him; but Mr. Harrington never found it convenient even to start on the preparation of his share thereof, consequently the progress was slow and unsatisfactory during the eight years of the firm's existence; and a great deal of what the author succeeded in writing became stale and antiquated, owing to the progress that was continually being made in the science of bridge building.

In the summer of 1914, while the author was in Cuba struggling with the materialization of a great bridge project, Mr. Harrington announced his intention of withdrawing from the firm; and, of course, the author agreed, reserving later the right of twelve months' notice according to the terms of the partnership contract. When Mr. Harrington and he met about the end of October, it was decided that the author should take over the writing of the proposed treatise, but should deal in a general way only with the subject of movable bridges, so that Mr. Harrington may some day write an exhaustive and detailed monograph thereon, as he is eminently capable of doing. It is to be hoped that nothing will prevent his undertaking and completing the proposed book on the lines now contemplated; for it would certainly be a great boon to bridge builders.

Had it not been for the European War and the consequent utter paralyzation of bridge engineering due to its baneful influence, not only in Europe but also in the United States—and in fact throughout the world—this book could never have been written. Recognizing that the inactivity in bridge work would probably continue as long as the war lasts, and that the end thereof was likely to be remote, the author decided not to attempt the disheartening task of endeavoring to secure small business for the sole purpose of keeping occupied those of the firm's assistants who had chosen to cast in their lot with him, but to settle down systematically

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to work on the preparation of the MS. of his book and to continue it without cessation until completion. He was not able to make an actual start on it until the first of December, and his assistants were not in shape to give him much aid until January or February, owing to the necessity for completing the plans of the Pacific Highway Bridge over the Columbia River between Vancouver, Wash., and Portland, Ore., on which the firm of Waddell and Harrington had been retained as engineers.

The first step taken was a rearrangement of the chapters and a combination of certain of them, thus reducing the total number from one hundred and six to eighty without omitting anything but one chapter on "Ocean Piers" and the contemplated detailed treatment of shopwork, substituting, though, for the latter a chapter on "Shopwork as Affecting Bridge Design." The next step was a drastic one, viz., the rewriting personally by the author of some forty chapters, representing his entire intermittent labor on the book during a period of eight and a half years, including all seven of the finished specifications for designing and construction, which were combined mainly into two chapters, viz., LXXVIII and LXXIX. This involved an immense amount of labor; but there was taken at the same time a still more drastic step involving much more, viz., the changing of the designing specifications so as to bring them not only up to date but also a trifle ahead of the times. The changes, it is true, were by no means radical, but they involved live loads, impacts, and intensities of working stresses, with the consequence that the old record diagrams, considerably over one hundred all told, had to be revised to meet the conditions of the new specifications. Their number, however, was materially reduced by combination, care being taken not to confuse the user thereof by any too complicated methods of recording. Many new diagrams were also prepared so as to systematize certain data that theretofore had been deemed by the office force too complicated to be susceptible of diagrammatization. All this necessitated extensive labor on the part of both the author and his assistants. Again, it was found necessary to make a number of special investigations in order to determine certain formulæ, functions, and relations previously undeduced or else either unsatisfactorily or too approximately established. All this has kept very busy everybody concerned, the author working regularly eleven or twelve hours per day (except Sundays, when only seven hours were utilized) and his assistants from eight to ten hours per day. The principal of these assistants were Messrs. Robert C. Barnett, C.E., Herman H. Fox, C.E., Shortridge Hardesty, C.E., N. Everett Waddell, C.E. (the author's son and now his partner), and Miss A. C. Humbrock, his stenographer. His brother, Robert W. Waddell, C.E., also aided by compiling a portion of the Glossary. The author's preliminary estimate of time required for the preparation of the MS. has been exceeded by one hundred and fifty per cent, and the anticipated labor and expense have been more than doubled. As a matter of interest to those engineers who do not indulge in the luxury of technical-book writing, it might be stated that the total cash outlay involved in preparing the MS. for the publisher and in doing the proofreading amounts to $13,000, including the money spent by the old firm in making a thorough search of engineering literature, which expense, of course, the author assumed in taking over the writing of the book. To this amount must be added at least $10,500 to cover the cost of getting out the first thousand copies, making a total expenditure of fully $23,500. This cost, perhaps, is excessive for engineering writing; and, of course, it could have been materially reduced by the author's making many of the computations himself; but such is not his practice, for he is a firm believer in the principle that "it is uneconomical to do yourself what you can pay another to do for you." Again, it might be observed that to prepare the MS. for such a treatise as this entirely unaided would require all of ten years of the author's undivided time and attention, and that the book when finished would then not be up to date. One of the most difficult tasks encountered has been continually to inject into matter deemed to be already complete new material due to the latest developments in engineering practice, amplifications of ideas previously covered quite thoroughly, and additional tables and diagrams specially prepared so as to bring the treatise not only up to present engineering practice but also somewhat in advance thereof.

In writing this book it has been the author's aim to give to his readers, concerning every branch of bridgework, all the information that he has been able to accumulate during a practice of forty years. Nothing of any value has been omitted, except such matter as can readily be obtained from other books; because he never has been a believer in the pseudo-economic idea that what has cost much labor and money to discover and record should be utilized only for one's personal gain. On that account there appear for the first time in print all the diagrams of weights of metal, quantities of masonry, costs of constructions, economic functions, etc., that this book contains.

As was the case when *De Pontibus* was written, it has been the author's endeavor to keep quite close to his own practice in the methods of bridge designing described; but as this work attempts to cover essentially the entire field of bridge engineering (excepting only the theory of stresses and similar matter which can be found in all standard books on bridges) while *De Pontibus* did not, it has become necessary in the illustrations to include occasionally structures designed by other engineers; and in all such cases full credit has been given them. It seems hardly necessary, but yet may be advisable, for the author to apologize to his readers for the characteristically personal style of his writing; and he herewith does so with the hope that he will be pardoned therefor, as he undoubtedly was in the case of *De Pontibus*, which was written and illustrated in exactly the same style and manner. The book is intended to be, in a certain sense, a record of the author's life work, prepared after a ripe experience and before age has begun to diminish his energy or to deteriorate his mental capacity. He feels that he has given to the profession which is so dear to him, and which he appreciates so highly, the best effort of which he is capable, trusting that its members will pardon his shortcomings, and that they will agree that when his time comes to pass on to the beyond, he will be worthy of that famous Colorado epitaph, with which he once concluded an address to a large body of engineering students, "He done his level damnedest; no angel could do no more."

Those readers who have perused *De Pontibus* will notice that a portion of the contents of that book has been absorbed in this one; but everything thus utilized has been brought up to date. Wherever any changes seemed advisable they were made, but otherwise the old text was copied *verbatim*. This was unavoidable if the new book were to be made complete, because there are certain facts and principles given in the old one that are permanent and unchangeable; and it would have been a serious mistake to omit them simply because the author had put them in print before. He has also quoted freely in certain places from papers that he has presented to technical societies, when such papers or portions of them expressed exactly what he desired to state. No apology for this is necessary. It will also be noticed that a prominent feature of the treatise is the comparatively small number of quotations from other writers, the author generally preferring to state his own opinions and conclusions directly. However, when it appeared advisable for him to depart from this practice, he did so without hesitation.

The method of numbering the various illustrations and the tables scattered throughout the book needs some explanation. Considerable thought was given to the suggestion that all these be grouped near the end thereof before it was decided to place them in the text as close as practicable to where they are first mentioned. The method of nomenclature adopted is to give to each illustration or each table the number of the chapter in which it appears, followed by a letter of the alphabet indicating its position in that chapter. Where more than twenty-six illustrations are contained in a single chapter, the letters are doubled, up to a total of fifty-two, after which they are trebled. Thus Fig. 19*j* shows that the illustration pertains to Chapter XIX and that it is the tenth given therein. Similarly, Fig. 55*ee* denotes that the illustration thus named belongs to Chapter LV and that it is the thirty-first in order. Had the number been 55*eee*, it would have indicated the fifty-seventh illustration of Chapter LV. Of course, there will never be any need for knowing the number corresponding to the lettering, as the latter is intended only as an aid in finding the location of any required illustration or table by turning over quickly the pages of the chapter to which it belongs. Attention is called to the fact that in listing the various illustrations, they have been divided into three groups, viz., "Ordinary Figures," "Cross-Section Diagrams," and "Views." It is thought that this division will aid the reader in locating any particular illustration to which he desires to refer.

In writing a preface to a technical work, custom decrees that it is permissible to state what classes of persons can use it to advantage and how; and the author desires to avail himself of that privilege.

Primarily the book should prove useful to all engineers who are engaged either directly or indirectly in the designing and building of bridges, and especially to the younger ones; for not only are the principles of design explained and exemplified, but also many practical hints are given, which otherwise could come to them only through wide experience. With the various tables and diagrams it is feasible to make quickly a close estimate of cost for nearly every kind of bridge and for structures of any length and size yet attained, no matter what may be the complication of traffic that they have to carry. Again, in respect to spans of unprecedented weight and length, data are given for determining, at least approximately, the weights of metal required by the use of alloy steels of various elastic limits. Also the practical treatment of secondary, temperature, and indeterminate stresses is expounded, as well as are the standard methods of computing for deflection and proportioning for camber. The general detailing for all kinds of fixed spans is treated in connection with the first principles of designing. It is true that the special detailing of movable spans is not covered, except incidentally; nevertheless the same general principles will apply to these structures. The protection of metalwork is dealt with at length, and its importance is emphasized. The practice of the designing and construction of reinforced-concrete bridges is explained very fully, but none of the theory is given, excepting only a small portion in relation to certain formulæ that have been established in the author's office. All kinds of substructures are described and illustrated, specifications are given for their designing, and explanations of how, when, and where to adopt the different types are furnished. The preliminary work antecedent to the actual designing of bridges receives thorough attention in Chapters XLVI to LI, inclusive, and also in Chapter LIV. Aesthetics and true economy in design are fully discussed in Chapters LII and LIII. Quantities of materials of the various kinds used in bridge construction are given in Chapters LV and LVI; and the preparation of estimates, specifications, contracts, and reports is treated at length subsequently. In Chapters LX to LXV, inclusive, all matters relating to field engineering are covered; and the inspection of materials of all kinds is exhaustively discussed in Chapter LIX. Business matters relating to engineering receive attention in Chapters LXXI to LXXV, inclusive; and questions of an ethical nature are dealt with in Chapters II, LXXVI, and LXXVII. In Chapter LXXVIII are given in complete detail specifications governing the designing of the superstructures for all kinds of bridges, trestles, viaducts, and elevated railroads, together with a clause-index at the end for the use of computers, so as to enable them to find quickly any particular clause required. Specifications for the designing of substructure are appended to Chapter XLIII, and specifications for the designing of reinforced-concrete bridges will be found in Chapter XXXVII. In Chapter LXXIX are complete specifications governing the manufacture and erection of the superstructure, substructure, approaches, and all accessory works of bridges, trestles, viaducts, and elevated railroads; and in addition thereto is a clause-index that makes the specifications easy to use. By employing them in the manner explained, any bright young engineer who has a general knowledge of bridgework will be able to prepare truly first-class construction specifications, complete in every particular and systematically arranged, when calling for bids upon any class of bridgework for which the preliminary drawings have been made, and quantities of materials computed. Finally, in Chapter LXXX will be found the most exhaustive glossary of technical terms used in bridgework that has ever been compiled; and the general index which closes the book is so complete as to enable any one to find very quickly any point whatsoever that is dealt with in the treatise.

While the work was not prepared as a text-book for engineering students, it is well adapted to supplement the standard treatises used in the classroom. It would be of value to them as a book of reference; and if there were three or four copies in the library, they would be found generally instructive on such matters as the history of bridge engineering, ethics, materials, loads, intensities, first principles of designing, æsthetics, economics, and business, all of which subjects are treated in a manner that is simple and which makes easy reading; consequently those students who are studying for the sake of learning and not merely to secure a degree with the least possible mental effort would read such chapters, especially if they were advised to do so by their instructors. Again, in the preparation of thesis work and in the making of students' designs the more solid chapters would be found a great help, giving, as they do, a vast fund of practical information such as the designer needs.

The book should be found useful by those higher officials of the railroads who are not engineers; for the presidents, general managers, and superintendents ought to be able to estimate on the costs of bridges for their systems; and they could readily do so by utilizing the diagrams given in Chapters LV and LVI. Besides, there is much information of a general nature scattered throughout the book, which would be of interest to such men.

As a book of reference for the general public, the work should find a place on the shelves of public libraries, especially in those centres of population where much bridgework is done, and in those where engineering students congregate; and it certainly ought to prove useful in the libraries of all universities, colleges, and technical schools.

It is evident, as previously indicated, that this book of a million words with all its diagrams, tables, and formulæ could not have been written in sixteen months by one man working single-handed, no matter how large might be the amount of his accumulated data. On the contrary, it was necessary to employ constantly a large number of men, all working, of course, under the close personal supervision and direction of the author, so as to prepare or digest the material needed for his use in writing. To these gentlemen, and especially to Messrs. Hardesty, Barnett, Fox, and Everett Waddell, the author is greatly indebted for their faithful and intelligent aid and painstaking care; and he herewith tenders them his hearty thanks therefor, trusting that the experience they have thus obtained in technical-book writing will stand them in good stead in future years. He desires specially to thank his stenographer, Miss Humbrock, for her careful work and unfailing willingness and courtesy in typing and retyping the chapters and in modifying them from time to time by changes, insertions, and additions that proved to be necessary as the work progressed.

Throughout the book are to be found acknowledgments with thanks for aid received from brother engineers, both directly for this treatise and through their published works; but the author desires to repeat here his thanks to his old friends, Messrs. Henry W. Hodge, Paul L. Wolfel, Albert Reichmann, and Hildreth & Company for the trouble they took to furnish certain valuable data for which they were asked. To Dr. Charles Warren Hunt, Secretary of the American Society of Civil Engineers, are tendered the author's thanks for his courtesy in having a search made in the Society's library for data on a number of subjects. To the Teachnor-Bartberger Company of Kansas City, which prepared the illustrations, is rendered a willing acknowledgment of obligation for their excellent work, and especially to their Mr. Roger Cunningham for much valuable advice as to how best to prepare the diagrams so as to obtain fine prints without going to the practically prohibitive expense of making wax cuts. In preparing the Glossary of Technical Terms, reference has been made to a number of illustrations in Prof. Ketchum's invaluable work, "Structural Engineers' Handbook," instead of reproducing them in this already too elaborate and expensive treatise. No apology for this *petite économie* is necessary, because any engineer who is sufficiently interested in bridgework to warrant his using the said glossary cannot afford to be without the said handbook. Finally, the author desires to acknowledge his indebtedness to the American Railway Engineering Association (in his opinion, the most active and efficient engineering society in the United States) for much valuable data taken from its *Proceedings*.

In conclusion, the author would state that he considers this book to be the greatest and most important work of his entire professional career, which has been an unusually busy one; and most certainly he would be bitterly disappointed if, for many years to come, it should fail to prove of great value to the engineering profession, and especially to the younger members thereof, in whose success he has always taken a deep interest, primarily on account of his six years' association with young men when he was a teacher of engineering, and also because of the memory of his own hard struggle to attain professional success.

J. A. L. WADDELL.

Kansas City, Mo., May 17, 1916.

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BRIDGE ENGINEERING

CHAPTER I

EVOLUTION OF BRIDGE ENGINEERING

TODAY bridge building is truly a science; only three decades back it
was hardly worthy to be termed an art; while seventy-five years ago, in
ur own country at least, it was no better than a trade. Nearly all of
he important and distinctive features of modern American bridge prac-
ice have been developed within the memories of engineers still living;
nd so far as most lines of bridge construction are concerned, the same
atement holds true for European practice as well. But while bridge
uilding as a learned profession is thus of very recent origin, it must not
e thought that the previous centuries made no contributions to our
nowledge of bridge construction; for there are in existence today bridges
hat have withstood the ravages of time for over two thousand years,
nd the records of antiquity tell of others built many centuries earlier—
ven before the dawn of authentic history. But bridge engineering reaches
till farther back into the past; for primitive man must have built many
rossings over shallow streams by piling in rocks for piers and covering
hem with slabs of stone or logs, or by felling trees so as to span small
ivers. However, we must look to still earlier ages for the beginning—
ack to the days when our arboreal ancestors formed living chains of their
wn bodies, holding to each other with arms, legs, and tails, thus con-
ructing suspension bridges across the water from the overhanging
nches of opposite trees, in order to let their tribe pass over in safety
the other side, in the same manner as is still practised by their unde-
loped descendants who reside today in the South American forests,
shown in Fig. 1*a*. Assuredly, the aged simian of those bygone times
ho directed the construction and operation of such a structure was a
dge engineer in the truest sense of the word as well as a being of high
telligence in comparison with his contemporaries.

From such a beginning to the present-day achievement of an East
iver suspension bridge or a Quebec cantilever structure is, indeed, a long
vance; but could we trace the intervening steps of development, we
ould find that our modern bridge is the cumulative result of the past
orts of the bridge constructor to meet the increasing demands on his
genuity. To give the reader a better conception and appreciation of
e magnitude and character of this advance, and of the factors that
ve influenced it to a large degree, it is well to review in a brief way
ch facts as history presents or that can be gleaned from observation

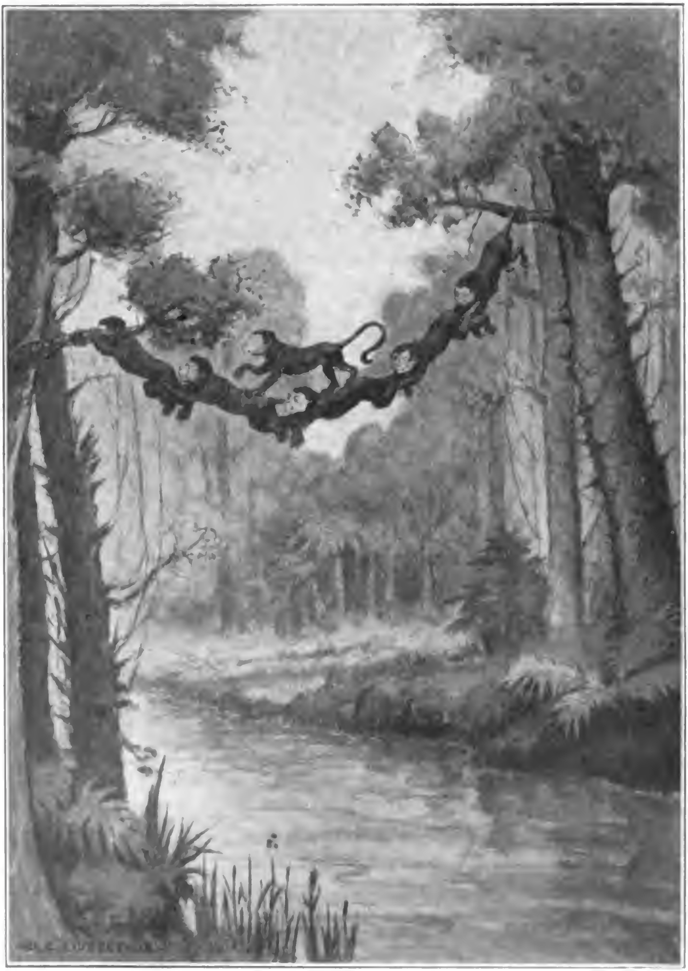

Fig. 1*a*. Monkey Bridge.

of existing structures or the ruins of early ones. From such data it may be seen that the evolution of bridge engineering is the resulting combination of the evolution of the form of structure, of the materials of construction, of the methods of design, of the methods of fabrication, and of the methods of erection. There has been a collateral development along these several lines, and the history of one involves that of the others, so that we find it necessary to pursue one line to a certain stage of development and then revert to the beginning and trace out the evolution of another branch. Furthermore, as bridge construction has been practised contemporaneously in the various parts of the inhabited globe under different environments, we should naturally expect the early development to have gone forward along different lines in these several portions, and such we find is the case; hence there will be an advantage in following independently, for a time, each line until it converges with the others.

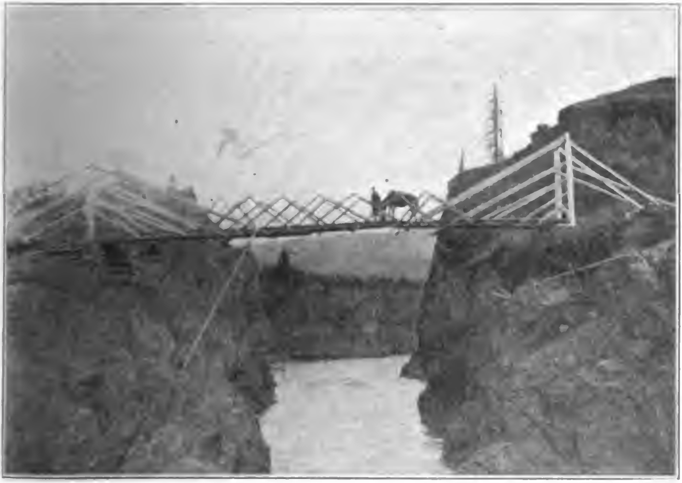

FIG. 1*b*. Indian Bridge over the Bulkley River at Moricetown, B. C.

Considering first the evolution of the form of structure, we must seek for the earliest types among primitive man in prehistoric times. Of these we have, of course, no records and can judge only from the work of savage races of our own day. Undoubtedly the prototype of our present beam or girder span was the log or tree felled across a stream, while the "monkey bridge," or a hanging, looping vine furnished the inspiration for the early suspension bridge. That our primeval ancestors may have built structures of some magnitude may be inferred from two bridges constructed by the British Columbian Indians, photographs of which are shown in Figs. 1*b* and 1*c*. The former is a view of a cantilever bridge across the Bulkley River at Moricetown, about fifty miles above Hazleton, B. C. The span length is about 75 feet and the height above the water about 100 feet. Two poles were pushed out from either side and anchored down with heavy stones, and between the opposite ends of the two pairs of cantilevered girders thus formed were placed two overlapping logs all lashed together with telegraph wire.

Fig. 1*c* pictures the Ahwillgate Indian Bridge at the village of Ahwillgate across the same stream about four miles above its junction with the Skeena River at Hazleton. The span is 150 feet in the clear, and the height above the water is about 200 feet. The structure is entirely of Indian design and construction and was at one time a suspension bridge, the cables for which were made from telegraph wires twisted together. The timbers are all round poles lashed to one another with wire. There are two separate systems of suspension: first, from the wooden towers to and under a cross-log at the centre of the span; and, second, from the said towers to the tops of the vertical posts in the triangles, and thence to and under the cross-log just mentioned. The truss in the centre was an afterthought, having been added so as to stiffen the central portion of the structure. The tension members of the trusses (invisible in the photograph) are of twisted telegraph wires tightened by a Spanish windlass. There are wire guys from bridge to shore above and below to take

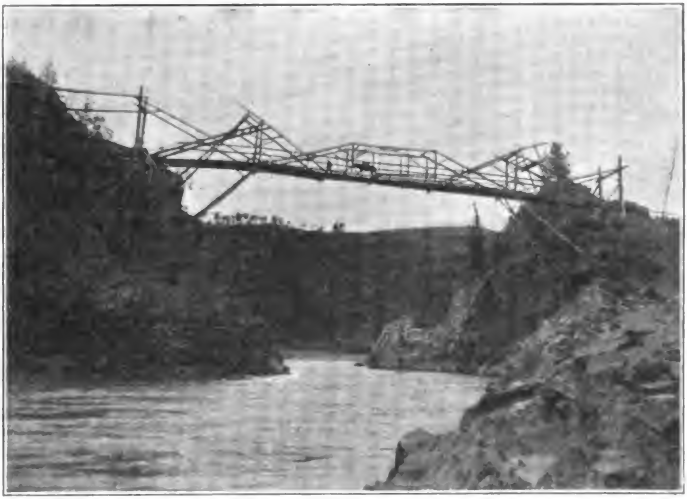

FIG. 1*c*. Indian Bridge over the Bulkley River at Ahwillgate, B. C.

up the wind pressure. The roadway is only six feet wide, being designed solely for the passage of pedestrians and pack-horses. The wire employed was left in the country when the "Collins Overland Telegraph Line" was abandoned in 1866 on the completion of the Field Submarine Cable. This bridge existed as a cantilever in the early sixties before the wire was available. Fig. 1*d* shows quite clearly the details of construction of this primitive structure, which, undeniably, is a most creditable piece of work for entirely uneducated men. It proves that there are good bridge engineers outside of the civilized peoples of the world, and that constructive ability is not always confined to those who have learned to read and write. In corroboration of the implied suggestion that the structure just described is the work of uncivilized man, it might be stated that the tribe of Indians who built it would not permit it to be used for traffic until after it had been tested thoroughly by placing upon it a heavy load of squaws. Undoubtedly, they must have considered the advisability of making the test with horses or cattle, and have concluded that it would be more economic to risk losing their less valuable live stock.

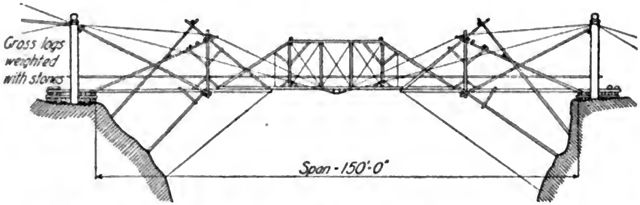

FIG. 1*d*. Indian Bridge over the Bulkley River at Ahwillgate, B. C.

Passing from the realm of conjecture and inference to that of a partially known and indefinite history, we find that the Caravan Bridge over the River Meles, at Smyrna in Asia Minor, is of a very early though unknown date, and is believed by many to be the oldest existing bridge. It is a single span, forty feet in length, and still in use, most of it being in its original condition. There is a very ancient beam-type bridge in England, which is believed to be of the same age as Stonehenge, or over two thousand years old. It is over the East Dart, and has three piers constructed of granite blocks which carry granite slabs, one of them being fifteen feet long by six feet wide.

The earliest bridge of which there is any truly authentic record was built over the Euphrates at Babylon by Semiramis or Nitocris about 780 B.C. Herodotus described it, writing in 484 B.C. It was a short-span structure thirty-five feet wide, of timber beams resting on stone piers. Only a few other ancient bridges, except those that were built by the Romans, can be described with any certainty. Two other early bridges of the beam or girder type were built in Greece about 425 B.C., one at

Assos and one at Eubœa. Both had stone piers, which carried stone lintels or beams in the case of the bridge at Assos, and timber beams in the case of the one at Eubœa.

However, the Romans were the real bridge builders of antiquity, and the records of much of their work are fairly well preserved. The earliest Roman bridge of which we have any exact information was the Pons Sublicius, over the Tiber at Rome. This was a timber structure, of the beam type, resting on piles and so arranged that the floor could be removed. It was built about 620 B.C., and was made famous by Horatius Cocles holding it against the Etruscans under Lars Porsena in 598 B.C. The most celebrated of all the early bridges was Cæsar's pile trestle, built in ten days' time, over the Rhine, during the year 55 B.C. Under the Romans the timber beam span reached its culmination. No great development could take place in this type of structure until a correct theory of beam action had been established and a material obtained that would meet the requirements of such theory. The passing from a beam of rectangular section to that of the more efficient I section was of rather late but unknown date—after cast iron became available for bridge construction. This, of course, was superseded by the rolled section or a beam built up of plates and angles—the modern girder.

Another early form of construction used by primitive man was the suspension type. It is more than likely that suspension bridges of crude form were the first kind of bridge to be employed for the spanning of openings which exceeded the length of a single log. Tyrrell in his "History of Bridge Engineering" states that they were used in remote ages in China, Japan, India, and Tibet, also by the Dyaks of Borneo, the Aztecs of Mexico, and the natives of Peru and other parts of South America. The cables of these primitive structures were made of twisted vines or straps of hide and fastened to trees or other permanent objects on shore. No date can be assigned for the building of the first suspension bridge; one of 330 feet span is said to have been built in China about A.D. 65, and it is believed that others had been completed in that country many centuries earlier. No very great span-length could be attained until stronger materials could be had for the cables, so that it remained for comparatively recent centuries to see much development in this type. Iron chains for suspension cables were adopted in both India and Japan five hundred years or more ago, while rope was employed for the same purpose in Europe, India, and South America several centuries back. But little improvement was made until modern times when the stiffening truss was added and wrought iron and steel were made available for construction.

Another early form of bridge was the cantilever span. As this required a little higher order of intelligence to construct than the beam or the suspension type, it is likely that its advent was of a later date. As far back as 1100 B.C. it is known that the ancient Greeks employed the "corbeled arch," which, strictly speaking, is a form of cantilever construction and not an arch at all. The Egyptians made use of corbeled stone arches two or three thousand years before the Greeks. One of the ancient examples of cantilever construction is that of a Japanese structure known as the "Shogun's Bridge" in the sacred City of Nikko, which bridge was erected about 500 or 600 A.D. It has stone piers or, more strictly speaking, bents; for the cylindrical columns are pierced with rectangular holes to permit of the insertion of tightly fitting, cut-stone struts. The superstructure, which is mainly of timber, consists of beams jutting out from each pier, with the gap between their ends spanned by other beams, making true cantilever construction. As the author lived within a short distance of this bridge during two summers of his sojourn in Japan in the early eighties, he became quite familiar with its appearance, which is truly artistic—like most other Japanese constructions. Fig. 1*e*, reproduced from a photograph which he secured at that time, gives a rather inadequate representation of its æsthetic nature. The wooden portion of the bridge has since been destroyed by fire and replaced. Of course, the timber parts of this historic structure must have been renewed many times during the centuries that have passed since its first construction; but it is claimed, and probably with truth, that the stone bents which form the substructure are those originally built.

The Chinese are believed to have constructed cantilever bridges many centuries ago. A cantilever bridge built in 1650 A.D. at Wandipore, Tibet, had a span of 112 feet and lasted 150 years. It was a timber structure put together with wooden pegs, and without metal of any kind being used in the span. The Hindoos are also credited with having built cantilever spans at a very remote age. The development of the cantilever, however, did not proceed very far until modern times, when the truss form of structure had become established and when iron and steel constituted the materials of construction.

Another and later form of bridge and one requiring a higher degree of skill and intelligence than the other types referred to was the arch. The construction of masonry arches began before the days of authentic history; and it is impossible to determine to whom should be given the credit of building the first bridge of that type. It is likely that the previously mentioned "corbeled arch" such as used by the ancient Greeks was the precursor of the true arch. As before stated, corbeled stone arches were used by the Egyptians in the Pyramid of Gizeh, dating back some three or four thousand years before the Christian Era; and brick arches of crude form are found in the ruins of Thebes in structures that were probably built about 2900 B.C. Before the founding of Rome, the Etruscans in Italy had used, quite largely, arches of the corbeled type, and occasionally the true arch; and the Romans doubtless drew from them their early knowledge of that style of construction. A true stone arch was found in a tomb in Thebes, which tomb is thought to have been

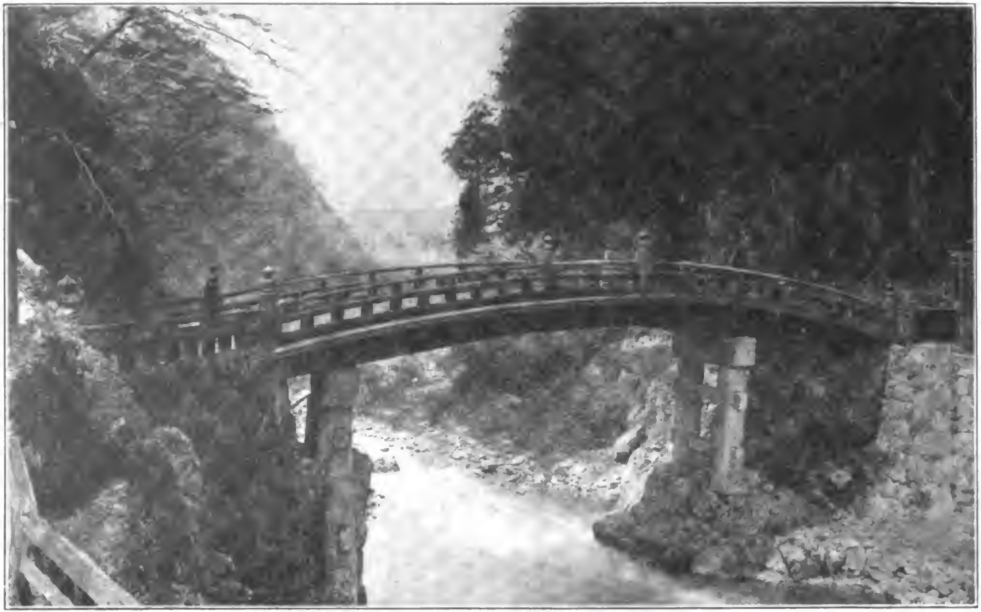

FIG. 1*e*. "Shogun's Bridge" in The Sacred City of Nikko, Japan.

constructed about 1540 B.C. But all of these may have been antedated by a true stone arch found in a pyramid of red sandstone on the island of Meroe, in Ethiopia. Some authorities consider this to be the earliest arch known and that the Egyptians obtained their knowledge of arches from the Ethiopians. All of the arches above mentioned are of short span; and it is not certain that either of these peoples applied the arch to bridge construction. It is very unlikely that the Egyptians ever built any large spans of that type, as they mistrusted the arch, saying that "it never sleeps"; that is, they believed that the horizontal pressure on its abutments would eventually accomplish its destruction. The Hindoos, too, have always refused to adopt the arch, saying, as did the Egyptians, that "it never sleeps." Because of this, as has been previously noted, they constructed suspension bridges many centuries ago; and they are believed to have built cantilever structures as well.

The inhabitants of the valleys of the Euphrates and the Tigris also were familiar with the arch at a very early period. The Babylonians built pointed brick arches for sewers certainly as early as 1300 B.C., and some of them are believed to date back to 4000 B.C. In the time of Nimrod, about 2200 B.C., the River Euphrates in the City of Babylon was crossed, it is claimed, by a single brick arch thirty feet wide and six hundred and sixty feet long; but this information must be taken *cum grano salis.* It is more than likely that the total length of the arch bridge was six hundred and sixty feet, and that the spans were short, because there is a record of another Babylon bridge of just that length, composed of stone piers supporting a wooden platform, as previously described. This structure may very well have been the first large arch bridge ever constructed.

The Chinese have employed the true semi-circular arch for ages, and have brought its construction to a very high plane of excellence, although their old spans were always short. Arches were built in their Great Wall about 214 B.C., but the time when they were first introduced in China is unknown. Chin-nong, who is supposed to have lived about 2900 B.C., is said to have constructed bridges over navigable streams, but their type is not stated, and the date is very uncertain. It may be that to him, and not to the Babylonians, belongs the honor of having constructed the first large arch bridge.

However, it remained for the Romans to bring the masonry arch to its high degree of development during the eleven centuries succeeding the construction of Pons Sublicius previously mentioned. Many notable bridges, which were built by them in this period, are characterized by the semicircular masonry arch. Tarquinius Priscus is reputed to have constructed a three-span bridge of this type, known as Pons Salarius, over the Teverone, as early as 600 B.C. Probably the first stone arch bridge over the Tiber at Rome was Pons Æmilius, built about 178 B.C. on the site of the modern Ponte Rotto. It was followed in 100 B.C. by a similar structure called Pons Milvius, which exists today under the name of

Ponte Molle, and contains portions of the original construction. Pons Fabricius, a masonry arch bridge of two eighty-foot spans, also over the Tiber at Rome, was built about 62 B.C. This still exists under the name of Ponte Quattro-Capi, and is yet in use, with nearly the entire structure in its original condition. In all, the Romans built eight bridges over the Tiber within the City of Rome, concerning which structures our knowledge is certain, and there are evidences that several others existed as well; and many other bridges were built at various places throughout the Empire. Some of these were portions of the magnificent system of stone roads which ran in all directions from the Eternal City, while others were temporary bridges constructed for military purposes only.

A large timber arch bridge was the one over the Danube in Hungary, which was constructed by the order of the Emperor Trajan in 104 A.D. It was designed and built by Apollodorus of Damascus, the greatest engineer of that period, and contained twenty wooden arch spans resting on cut-stone piers. There is good reason for believing that the length of each span may have been as much as 170 feet.

The Romans constructed also many arched sewers, some of which are still in use, and many aqueducts, some of which were carried over valleys and streams on large masonry arch bridges. One of the best known of these aqueducts is the Pont du Gard, built in 19 B.C., to supply the city of Nîmes in France with water; and it is still used for that purpose. It is in three stories, and has a length of 885 feet, the greatest height being 160 feet.

The Romans brought the art of constructing the semicircular stone arch to a high degree of perfection. They employed cut-stone voussoirs, fitted together without mortar; and so exactly was the work done that they appear to have been ground. A hydraulic cement of a pozzuolanic nature was employed for the making of concrete, which was utilized for backfilling arches, laying up walls, lining aqueducts, and many other purposes. A great deal of it is nearly perfect after two thousand years. The arches were usually of short span, but some of them were about one hundred and twenty (120) feet long, comparing very favorably with similar bridges of the present day. Their subaqueous foundation work was not so good, as the piers generally rested on stones piled up on the river-bed, which was not always excavated previously; and their width was usually about one-third of the clear span. On account of the resulting contraction of the stream and the unsatisfactory foundations, the piers were frequently undermined. In some of their structures, the spandrels over the piers were pierced with small arches, in an effort to increase the waterway.

With the fall of Rome, bridge construction in Europe came to a stop, and for many centuries little progress was made. In consequence, the magnificent system of roads and bridges of the Roman Empire soon fell into decay, very few bridges being built, and nearly all of these being poorly constructed. Probably the pioneer bridge builders of the Middle Ages were the Moors in Spain. In the twelfth century the Benedictine monks founded a religious order known as the "Brothers of the Bridge" (*Fratres Pontis*), the duty of whose members was the construction and repair of bridges; and under their leadership considerable work was done. Their substructure work was notably better than that of the Romans. There were a few other notable contributions to bridge building made during this period. The Gothic or pointed arches appeared about the thirteenth century, and the segmental and elliptical arches about the same time. The Gothic arches were never widely used for bridges, to which their outline is ill adapted; but the other types mentioned soon found much favor, and even today they are employed for many structures. The segmental type was adopted in 1380 for the construction of a granite arch of two hundred and fifty-one (251) feet span and eighty-seven (87) feet rise over the Adda River at Trezzo, in Italy. It was of unprecedented size, and it was not until the opening years of the present century that a masonry arch of longer span was constructed. It was destroyed in 1410, but the abutments remained until recent years. The elastic arch, as distinguished from the voussoir arch, did not make its appearance until the early part of the nineteenth century. The first span of this type was the wrought-iron foot bridge designed by the French engineer Bruyère and built over the river Crou at St. Denis, in 1808. However, it was nearly the middle of the nineteenth century before the superiority of wrought iron was recognized and cast iron was discarded for bridge purposes. The development of the elastic theory gave a basis for rational designing, and the introduction of steel and reinforced concrete has led to the present high status of arch bridge building.

A somewhat later form than the beam, suspension, cantilever, or arch type, perhaps, is the pontoon bridge, for some skill in boat building must have been developed before pontoons could have been used for supporting the spans. The Chinese are believed to have built pontoon bridges, as well as cantilevers, many centuries ago, and to have provided means for opening some of the spans for passing vessels. Homer, who lived some time between 800 and 1000 B.C., writes as though bridges were common in his day, and mentions in particular pontoon bridges for the passage of armies. It is known that the Persian kings Cyrus, Darius, and Xerxes about 500 B.C. used pontoon bridges for military purposes, crossing in this manner the Hellespont, the Bosphorus, and even the Danube, and that Alexander the Great constructed similar bridges about 330 B.C. Very little change in this form of construction has occurred, as other types have proved more desirable for permanent structures.

The last form of bridge construction to be evolved, but the one destined to promote the highest development of the art of bridge building, was the truss. It remained for Palladio, an Italian architect of the sixteenth century, to invent several trusses (making use of the panel principle) which were quite similar in form to our modern types. He constructed many bridge and roof trusses of timber, and wrote an elaborate treatise on architecture, in which they were fully described. Unfortunately for mankind, his discovery was allowed to lie unnoticed for several centuries. A notable advance in bridge construction occurred about 1760, when John and Ulrich Grubenmann built several timber spans near Baden, Germany, the largest being one of three hundred and ninety (390) feet over the Limmat at Wittengen—the longest timber span on record. The members of these structures were arranged in a complicated manner that would defy stress computation. Prof. Wm. H. Burr, in his "Design and Construction of Metallic Bridges," describes it as consisting "more nearly of a superposition of a number of queen-post trusses with some timbers disposed throughout its length in such a way as to act somewhat like an arch."

So far attention has been directed to the development of superstructure forms, with but brief mention of substructure work. It has been noted that the subaqueous work of the Romans was inferior to that of their superstructure. Apollodorus, in the construction of the bridge over the Danube previously mentioned, employed some form of caisson for building the piers; while for some other structures foundation piles, driven until their tops were below low water, appear to have been used. They always tried to locate their piers so that the bases could be laid in the dry; and they preferred rock foundations. The foundations of London Bridge, begun in 1176 by Peter of Colechurch, consisted of strong elm piles, driven deep, with a timber platform thereon to support the masonry of the shafts. This work proved to be very substantial, and the piers of several other bridges were built in a similar manner. Gradually further improvements in substructure work were made, but it was not until 1635 that a dredging machine was employed for the first time in the construction of a bridge at Maastricht in Holland, and nearly a century later a saw was invented which would cut off piles sixteen feet under water. In constructing the piers for the bridge of the Tuileries, which was designed by Mansard and begun in 1685, Frère Romaine used a dredging machine to prepare the bed of the river, and then sank a barge filled with stones, afterward surrounding it with piles and a jetty. He then lowered into the barge a chest containing courses of stone cramped together.

In 1738, Labelye, in building Westminster Bridge over the Thames at London, used a similar type of construction. His work is better known, and he is generally credited with being the originator of the modern type of caisson. It appears to be uncertain whether he simply dredged away the soft mud in the bed of the river in preparing the foundations of his piers, or whether he drove piles, cut them off, and built thereon a platform of timber. The caissons were boxes with watertight bottoms and sides, and were sunk to the foundation prepared by one of the methods described above, with the tops always remaining above the water. The masonry was built up inside the caisson until it was above the water level, and then the timber sides were removed. Sheet piling was driven later to protect the foundation against scour, a pile-driver run by three horses being used for the purpose. The piers were undermined by scour after several years, hence it is not likely that they rested on piles.

We have followed in a brief way the evolution of the form of structure down to the nineteenth century, when men of scientific attainments began to study the subject. It will now be well to revert to ancient times and observe the various materials used by early man in his bridge construction and see the limitations imposed by them upon his efforts. Prehistoric man had at his command wood, stone, and fibrous plants. These he had to use in their natural condition, for he had no tools with which to fashion his material. For many centuries these substances continued to be the only ones available for bridge construction, while the improvements made, due to the gradual introduction of tools, were in the nature of refinements in workmanship. It is readily seen that, with a material like wood or stone, beam spans could not attain a very great development, *i.e.*, they could not be employed for long spans or for carrying heavy loads—nor could the suspension span built of vegetable or animal fibre surpass in any great degree the work of prehistoric man. The stone arch, because its form produces an internal compression and by reason of the adaptability of its material to resist that compression, admitted of greater development than the other types. Again, the invention of the truss form permitted timber to be used to greater advantage than it had ever been before. With the introduction of iron into bridge construction, a larger field was first opened up for the suspension type of span, while the new material had little effect upon arches, beams, and cantilevers until later when it and its derivatives became, in conjunction with rational designing, one of the most important factors in the evolution of the modern bridge. So important is the effect of this innovation that it will be given special consideration further on in this chapter. For the present it will suffice to note that there has been a reciprocal effect, an action and a reaction on each other, between the form of construction and the material composing it.

We shall next consider the evolution that has taken place in the methods of design and the very potent influence that this factor has had in the development of bridge building.

Throughout the centuries of bridge development thus far discussed, the designer and builder had relied entirely on his experience and judgment, there being no theory to which he could turn for aid. As a result, progress had been slow; and the bridges of the eighteenth century were little better than those built two thousand years before by the Romans, except in the foundations. But during the seventeenth and eighteenth centuries men commenced to study the phenomena of nature anew, and the development of modern science began. The fundamental laws of physics and chemistry were discovered, and a basis for rational engineering theory was laid. Gradually correct methods of design were evolved, leading naturally to the marvellous expansion of the last century. Galileo announced a law of stress variation in beams as early as 1638, but it was grossly incorrect. Hooke stated in 1678 one of the fundamental laws of mechanics of materials, that of the proportionality of stress and strain—"*ut tensio sic vis.*" Marriotte in 1680 proved experimentally that the fibres on one side of a beam were compressed and those on the other side extended, and assumed the neutral axis to pass through the centre of gravity of the section. Bernoulli in 1694 applied Marriotte's law to determine the deflections of beams.

A great impetus was given to scientific design in 1716, when the French Government organized the Department of Roads and Bridges (*Département des Ponts et Chaussées*), the first engineer-in-chief being Gabriel. A drawing school in connection therewith was started in 1747; and in 1760 it was enlarged to become the *École des Ponts et Chaussées,* the noted engineer Perronet being placed in charge. About this time the French assumed the lead in bridge engineering and held it for a long time; and from them came the knowledge and inspiration of the early English engineers. Parent in 1713 announced the equality of the compressive and tensile stresses, which for a uniform variation of stress located the neutral axis at the centre of gravity. Little attention was paid to his discovery, however, and in 1773 Coulomb stated it independently. Even after this its importance was not realized for many years. Navier put the design of beams on a firm basis in 1824, and developed quite fully the theory of their deflection. He assumed the equality of the moments of the tensile and compressive stresses about the neutral axis, which is true for symmetrical sections only. Professor Eaton Hodgkinson published in England the first correct treatment of beam designing at about the same time as Navier. Finally Saint-Venant, a pupil of Navier's, gave in 1857 a complete and accurate analysis of the strength of beams, both at the elastic limit and at the ultimate strength.

While the theory of beam action was being developed, other investigators were studying the resistance of columns. Euler developed his famous column formula in 1744, using Bernoulli's work as a basis; and Lagrange and Navier later extended his work. In 1840 Prof. Eaton Hodgkinson published some tests on cast and wrought iron columns and determined constants for Euler's formula, from which fact it is frequently known as Hodgkinson's formula. It has been considerably used, especially in Europe, but is strictly applicable only to very large values of the ratio of length to radius of gyration, and is not generally considered suitable for actual columns by American engineers. The formula now known as Gordon's Formula was published originally by Thomas Tredgold; but the empirical constants for it were determined by Lewis Gordon, using the Hodgkinson experiments just mentioned. This formula was later modified by Rankine; and in the changed form, known as the Gordon-Rankine Formula, it has been widely employed. It still finds some favor, and it has generally been regarded by American and English engineers as being more rational than any other that has been proposed, though a good many disagree with this view. Some thirty years ago Edwin Thacher and T. H. Johnson, both recognized American authorities in bridge engineering, pointed out that a right-line formula could be evolved to represent quite satisfactorily the average of a great number of authentic column tests; and while it is purely empirical in form, it has been adopted very generally throughout the United States on account of its simplicity. Column formulæ have also been proposed by Merriman, J. B. Johnson, Marston, and others, and while each has possessed some individual merit, none of them has been adopted to any extent. A great many tests and researches concerning columns have been made in the past, and some important ones are under way at the present time; and it is to be hoped that ere long their designing and detailing may be put on a more rational basis.

The theory of the masonry arch has received attention since the time of Newton, and many volumes have been written on the subject, but the results have never been altogether satisfactory. The theory of the elastic arch and its application to metal arch ribs has been developed since 1840. In 1879 Weyrauch demonstrated the four fundamental equations upon which the elastic theory is based, and these have formed the foundation for further treatment by subsequent investigators and writers. In 1890 the Austrian Society of Engineers and Architects conducted a series of elaborate tests on arches used in buildings and bridges; and an exhaustive report thereon was published in 1895. The general conclusion reached was that the theory of elasticity gives the only solid foundation for theoretic investigation of all arches. For the steel arch the agreement between the observations and the predictions of the elastic theory was particularly satisfactory. This theory has come into general use in the design of reinforced concrete arches.

As has already been stated, the truss idea and the use of the panel were discovered in the sixteenth century by Palladio. Little attention was paid to his work in Europe, and it remained for American engineers to develop rational methods of analysis in the first half of the nineteenth century. Palmer, Burr, and Wernwag built combination arch and truss bridges in the early years of the century, apparently with little appreciation of the stresses involved; and Town, Long, Howe, and the Pratts invented pure truss types soon afterward, but were unable to figure the stresses in the various parts. In 1847 Squire Whipple published at Utica, N. Y., a "Work on Bridge Building," which gave the analysis of stresses in trusses in a surprisingly complete manner, while Herman Haupt published independently an inferior treatise in 1851. Whipple's book laid the foundation for the rational design of simple trusses; but it has remained for writers of the present day to develop the analysis of continuous beams and trusses, and of indeterminate structures and similar problems. Most of the important contributions have come from American and German writers.

The study of secondary stresses has progressed much of late years, and every effort is being made to minimize their effects in bridges, especially in view of the increasing use of riveted connections. One of the most noteworthy features of the latest design for the Quebec Bridge is the adoption of the "K" type of trussing, which type is remarkably free from secondary stresses. The secondary truss-members now in use in most American long-span bridges cause rather large secondary stresses; but it is practicable to eliminate them—at least in part. The calculation of these stresses is very tedious, and the resulting figures are rather uncertain. The best practice tends toward their reduction as far as possible rather than toward the making of any quantitative provision for them. A committee of the American Railway Engineering Association is studying this question at present, and has already made one valuable report upon it.

As previously stated, American practice used to pay no attention to æsthetics in bridge building; but conditions in this respect are rapidly changing. The extensive employment of reinforced concrete for short-span bridges makes their æsthetic treatment comparatively simple; because there is little excuse for building an ugly concrete bridge. Where steel construction is adopted, attempts are being made to obtain the best possible appearance, either by means of the arch (the ideal solution when practicable) or by polygonal top chords, which tend to produce a graceful effect. While the methods of design had been undergoing development there was a somewhat parallel development of material to meet the more and more exacting demands of rational design. For this reason the discussion of the evolution of materials which was previously interrupted will now be resumed. It has been pointed out how progress lagged until the introduction of iron into the construction, the first attempt being the use of iron chains in suspension bridges some five centuries ago, the employment of iron in the other types of spans coming much later. Although the manufacture of cast iron began in the fifteenth century, it was not until 1776 that the first cast-iron span was built at Coalbrookdale, England, over the Severn River. This bridge, which is still in use today, is an arch structure, the central span having a length of one hundred feet. It is composed of semicircular ribs made up of separate voussoirs. Quite a number of cast-iron bridges were built in Europe within the following one hundred years, nearly all of them being of the arch type. The brittleness of the metal rendered it an unsatisfactory material for bridges, and quite a number of failures occurred, especially under railway traffic. The first iron railroad bridge was constructed in 1823 on the Stockton and Darlington Railway in England. In 1847 when the Conway and Britannia bridges were to be designed, Robert Stephenson had a series of experiments on the strength of cast and wrought iron made by William Fairbairn and Eaton Hodgkinson, showing the great superiority of the latter material; and it was, therefore, decided to adopt tubular bridges of wrought iron rather than cast-iron arches. Within the next twenty years thereafter the use in Europe of cast iron for the main members of bridges practically ceased; but it was continued in America in important railroad structures a decade longer.

Iron was first rolled into structural shapes by Cord, of England, in 1783; and lattice bridges were first constructed there about 1824. In the earlier trusses the compression members were of cast iron and the tension members of wrought iron, but the former material gradually gave place to the latter. Wrought-iron bridges were usually built as arches or as bowstring or lenticular trusses (all modeled more or less after the stone arch), plate-girders, particularly of the tubular type, and multiple intersection or lattice trusses; but a number of suspension bridges were also built of this material.

In 1828 steel of the puddled variety was first utilized in bridgework for the eye-bar chains of the 300-foot suspension span at Vienna, Austria; but for many succeeding years its employment in bridge construction was practically discontinued. The Bessemer process for the manufacture of steel was invented in 1855, and the Siemens-Martin open-hearth process soon afterward. The development of the steel industry, however, was rather slow, and wrought iron remained the almost universal bridge metal until 1880; but between then and 1890 open-hearth steel came into vogue, supplanting entirely the wrought iron. Bessemer steel was never popular for bridgework on account of its lack of reliability, and especially because of its occasional tendency to crack under shop manipulation; nevertheless, strange to say, it was adopted for building the great Forth Bridge.

Natural cement was applied to bridge construction in the early part of the nineteenth century, and masonry work was much improved thereby. The development of the Portland cement industry, most of which has taken place since 1855, provided a more reliable material; and, as a result, plain concrete has come into very extensive use both for arch bridges and for the substructures of other forms of bridge construction. The introduction of Portland cement was responsible for the development of a new material of construction—reinforced concrete. The idea of increasing the tensile strength of mortar or concrete by embedding therein rods of metal or sticks of timber was not entirely new, as the Romans had used the former to a limited extent in the construction of roofs of tombs, and the Chinese had employed the latter in building the Great Wall. About 1840 attempts were made in Paris to construct floor slabs of plaster-of-paris reinforced with iron rods and bars; but the metal was found to rust rapidly. The reinforcement of Portland cement mortar or concrete by iron rods or wires was first proposed in 1850 by a French contractor, M. Lambot, who constructed a small boat in that manner. In 1854 an English plasterer named Wilkinson obtained a patent for a reinforced concrete floor construction; and in the following year M. François Coignet, a French contractor, was granted a similar patent. In 1861 Coignet proposed to build arches, beams, and pipes in this manner, and Monier, a Parisian gardener, about the same time began the construction of concrete tubs and tanks reinforced with wire. Both Coignet and Monier exhibited their work at the Paris Exposition in 1867; and in that year Monier patented his well-known system of reinforcement. For a good many years thereafter, however, reinforced concrete was but little employed in Europe. About 1879 Hennebique, in France, began building slabs of it, but did not patent his system of reinforcement until 1892. In 1880 Wayss bought Monier's German rights; and he and Bauschinger published tests on the material in 1884. In 1892 Melan developed in Austria the system which bears his name; and about the same time Möller in Germany and Wünsch and von Emperger in Hungary were beginning their well-known work. Methods of design which were reasonably rational were evolved, and reinforced concrete soon came into common use. In 1899 Considère published the results of a very important series of tests; and later investigators have expanded still further the knowledge of the subject. Today the material is used very widely in Europe for bridges of all kinds.

The advent of reinforced concrete has extended the development of beams and arches, bringing into common use the continuous girder and the hingeless arch; while the improvement in steel manufacture and the introduction of alloy steels have made possible the development of the truss type to its present high degree of effectiveness.

Steel-bridge construction in Europe today does not differ essentially from the standard American practice, although a few years ago the two were quite dissimilar; for pin-connected structures are almost unknown across the water, while until lately they have been the characteristic style of American construction. In Europe the double intersection and the multiple intersection trusses are still quite common, but in America the single-intersection truss has supplanted both of them. There is one feature of European, and especially of English, practice that is essentially different from the American, viz., that it favors the employment of much smaller truss depths, with the result of an absence of economy in both weight of metal and cost of structure, decidedly greater deflections under live load, and the general use of the objectionable pony-truss for short spans. The steel trestle, with its braced towers and alternate long and short spans, which is so characteristic of American railway construction, is rarely seen in Europe. One characteristic difference between the bridges of America and those of Continental Europe is that the latter are generally much more æsthetic, American engineers having had the bad habit of paying far more attention to economy than to appearance.

The evolution of European practice during modern times has been profoundly influenced by older forms, the masonry arch in particular. That type of structure itself has continued to be a favorite; and it has been used extensively for both short and long spans, the longest one being that of Plauen, in Saxony, with a span of 295 feet. A few departures from the older arch types are to be noted. The open-spandrel type has been employed to some extent for longer spans, with a consequent reduction in the load to be carried; and hinges have been used in several instances. These have been constructed of steel or granite, or have consisted merely of sheets of lead about eight-tenths (0.8) of an inch thick extending across the middle third of the rib.

Passing now entirely to the development of bridge engineering in America, it is to be observed that it has taken place along lines quite different from those followed in Europe. There have been no older types of structures to copy, and, as before stated, there has been very little attention paid to the æsthetic side. Evolution has proceeded mainly along the line of truss development with many varieties proposed and subjected to the test of actual use and an elimination of the impractical, uneconomic, and indeterminate forms. The increasing volume of railway traffic has been one of the main factors in causing the development. The crossing of America's numerous wide rivers has presented many problems quite different from those encountered in European practice; and, in addition, American engineers have almost always had to make every dollar go as far as possible. Much temporary construction has been necessary, owing to the lack of both time and money for permanent work.

Prior to 1840, most of the evolution of bridge construction in the United States was along the lines of the wooden truss and the wooden arch. The earliest important bridge built in this country was the "Great Bridge" at Boston, Mass. It was constructed in 1662 of timber superstructure supported by pile bents placed from fifteen to twenty feet apart. A few other bridges of the same type were built in New England prior to 1790. In the last decade of the eighteenth century, the building of long-span timber bridges began, Timothy Palmer being the most prominent constructor thereof. They were primarily arch spans, although there was also some truss action. In 1804 Theodore Burr constructed the bridge over the Hudson at Waterford, N. Y., with four spans varying in clear length from 154 to 180 feet. As it was effectively housed in, its timbers did not decay; and it lasted until 1909, when it was burned. During the latter portion of its life it carried heavy interurban cars. The most important member of the Burr truss was an arch rib; but there was also a very satisfactory all-wooden truss with a counter in every panel, somewhat like that exploited later by Howe and named after him. Several similar wooden bridges were built by Burr within the next few years; and in 1808 he constructed a wooden suspension bridge of four continuous spans across the Mohawk at Schenectady, N. Y., the longest span being 190 feet.

In 1812 Lewis Wernwag erected the "Colossus Bridge" over the Schuylkill River in Philadelphia, with a clear span of 340 feet. It was primarily a very flat arch with some truss action, and was the longest all-wooden bridge ever constructed in the United States.

In 1810 Thomas Pope proposed to cross the Hudson at New York with a 3,000-foot span and the East River with a 1,800-foot span, using his "Flying Pendant Lever Bridge," a fifty-foot model of which he constructed to illustrate his scheme. The proposed structure consisted of two immense cantilevers of timber extending out from massive abutments; but his idea was too far advanced for his time.

In 1820 Ithiel Town patented the Town lattice truss, shown in Fig. 1*f*, the first bridge truss essentially American. It was an all-wooden construction with a multiple intersection webbing that consisted of a multitude of light members; and it has been the prototype of many truss forms in both timber and metal. It soon became very popular, and many bridges were constructed from Town's plans, the greatest span length attained being 220 feet. The trusses were of uniform section throughout, and were proportioned by individual judgment rather than by analysis.

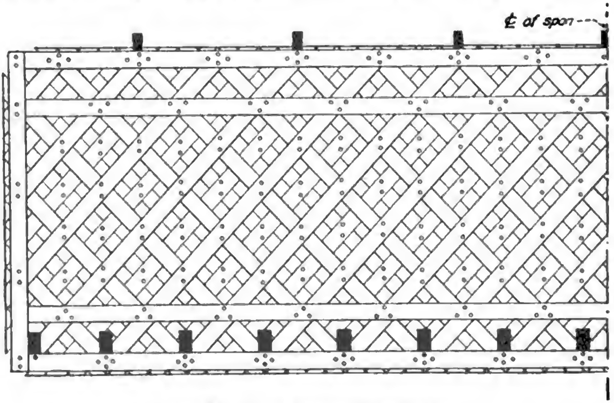

FIG. 1*f*. Town Lattice-Truss Bridge.

The Long truss was patented in 1830 and 1839; and its inventor, Colonel Long, published in 1836 a pamphlet describing it. He recognized clearly the true function of the panel counterbrace.

In 1840 William Howe patented the Howe truss, in which timber was used for the chords and the web diagonals and iron for the verticals. It grew in favor rapidly; and today it is recognized as the ideal truss form for wooden bridges. Its weak point is the bottom chord, which puts timber in tension and requires very elaborate splices.

In 1844 Thomas W. Pratt and Caleb Pratt patented the Pratt truss. It never attained much popularity for timber-bridge construction at that time; but during the eighties it was much used in "combination" bridges having all the compression members of wood and all the tension members of wrought iron or steel, notwithstanding its decided inferiority in respect to rigidity when built thus as compared with the Howe truss bridge. Its apparent advantage over the older competitor was a slight saving in first cost, and (when neither type was housed) a little longer life, the perishability of timber being more pronounced in tension than in compression members.

The first American suspension bridge of modern type, with horizontal floor suspended from the cables, was constructed about 1796 by James Finley. His cables were composed of wrought-iron loops or links, and his timbers were so framed as to give a rigid, continuous floor, which, in connection with the hand railings, constituted fair stiffening trusses. His longest span, 308 feet, was in the bridge over the Schuylkill River at Philadelphia, erected in 1809. This was replaced in 1816 by a 408-foot span carried by cables made of three-eighths-inch wires. It was built by White and Hazard, who owned a wire mill near by, and was the first wire suspension bridge ever constructed. A number of similar suspension bridges were built during the succeeding twenty years. In 1838 when Wernwag's "Colossus Bridge" at Philadelphia was burned, Charles Ellet replaced it by a wire suspension structure of 358 feet span; and in 1846 he built a 1,010-foot span suspension bridge at Wheeling, W. Va. All of his bridges were unstayed. About this time the famous American engineer, John A. Roebling, began to build suspension bridges. He used guys to brace his structures laterally, and also stays running diagonally from the tops of the towers to points in the floor, a practice that had been previously tried out and abandoned by the English. He also introduced the system of using wire cables in cylindrical form, wrapped with small wire, instead of separate wires side by side; and he inaugurated the practice of cradling the cables so as to increase the lateral stability of the structure.

The introduction of railroads in the United States came in 1829, and with it began the real development of bridge engineering. Wernwag built the first railway bridge for the Baltimore and Ohio Railroad at Monoquay in 1830. It was wooden and of the trussed-arch type. Many similar bridges were soon afterward constructed by Burr and others, and the Long and the Town lattice trusses were extensively employed; but soon after the introduction of the Howe truss in 1840 the latter became the standard for railway-bridge construction. The Pratt truss was used only to a limited extent by the railroads during the days of timber bridges. The first of the timber trestles which have played such an important part in American railway practice was built about 1840 on the line of the Philadelphia and Reading Railroad.

Thus far the construction of bridges had been carried out by men who were carpenters, and it was looked upon simply as a trade; but with the requirements imposed by the rapidly increasing railroad traffic, a higher class of bridgework was demanded. Furthermore, timber was being found inadequate for the work required of it, and a better material was being sought. As early as 1787, Thomas Paine had tried to introduce the cast-iron arch into American bridgework, but had failed. Wrought iron had been used in Finley's suspension bridges and in Howe's trusses; but the first patent for an all-iron bridge was taken out in 1833 by August Canfield; and in 1840 Earl Trumbull built the first structure of that kind over the Erie Canal at Frankfort, N. Y. It was a highway girder bridge of 77 feet span constructed mainly of cast iron with wrought-iron rods in a parabolic curve extending from end to end and supporting the cast-iron girders at various points, making the action very similar to that of a suspension bridge or of an inverted bowstring.

In the same year Squire Whipple built his first iron bridge, a bowstring girder with wrought-iron tension members and cast-iron compression members. He patented this type in 1841; but in 1846 he changed to the trapezoidal form.

The first iron railroad bridge in this country was designed by Richard Osborne, of the Philadelphia and Reading Railroad, and was erected near Manayunk, Pa., in 1845. It was a Howe truss structure of only thirty-four feet span, and had cast-iron compression members and wrought-iron tension members.

In 1846 and 1847 James Millholland built a tubular plate-girder bridge of wrought iron for the Baltimore and Ohio Railroad; and in 1846 Nathaniel Rider, of New York, patented a multiple intersection truss of combined cast and wrought iron. Several bridges of the latter type were built, but when one failed, about 1850, it was decided by most engineers to return to wooden bridges for railroad purposes.

Thus far the design of bridges was purely a matter of judgment and experience, for no method of analysis was known. As stated before, in 1847 Squire Whipple published at Utica, N. Y., a "Work on Bridge Building," and this marks the beginning of rational bridge design. Unfortunately, it was several years before his book became at all generally circulated.

The firm of Plymton & Murphy during the fifties made a model of a truss in which any member could be replaced by a spring balance in order to measure stresses. Before Whipple's book was widely known, the tendency of web members to fail near the ends of the span had been noticed, and in a few designs they had been made heavier than those near mid-span.

The decade from 1850 to 1860 marks a very important advance in

American bridge engineering, for the designing then came into the hands of educated engineers; and rational design really began. Benjamin H. Latrobe, the chief engineer of the Baltimore and Ohio Railroad, decided to use iron bridges in constructing the extension of his line from Cumberland to Wheeling. Wendell Bollman and Albert Fink, the inventors of the Bollman and the Fink trusses, were then working under him; and the first bridges of these types were constructed on that road in 1855, cast iron being used for the compression members and wrought iron for the tension members. In that year Fink designed some iron railroad trestles for the same line, using his patented truss for the spans.

In the same year the first of the double-intersection Whipple-truss bridges was built. It also was of both cast and wrought iron. During the same decade the Pennsylvania Railroad began the construction of bridges composed of Pratt trusses stiffened with arches, all compression members being of cast iron and all tension members of wrought iron.

The first pin-connected bridge was designed by J. W. Murphy for the Lehigh Valley Railroad in 1859. In the same year Howard Carroll built the first all-wrought-iron bridge for the New York Central Railroad. It was well constructed and of the lattice type. He employed iron track-stringers and floor-beams, and prepared a printed specification for railroad bridgework. J. H. Linville, of the Pennsylvania Railroad, first introduced wide, forged eye-bars in 1861; and in 1863 J. W. Murphy designed for the Lehigh Valley Railroad the first pin-connected truss bridge with all the main members of wrought iron, cast iron being used only for joint blocks. This bridge was of the double-intersection Whipple type; and, on account of the improvements introduced by Murphy, it has frequently been known as the Murphy-Whipple truss. In 1865 the first of the Post-truss bridges was constructed for the Erie Railroad, and that type was employed more or less thereafter for some fifteen years. They were generally built with cast-iron top chords and end-posts.

As before mentioned, a plate-girder bridge had been built for the Baltimore and Ohio as early as 1846. Another was constructed by the Pennsylvania Railroad in 1853, and one was designed in 1860 for the Boston and Albany by E. S. Philbrick. The only large tubular bridge ever built in North America was the Victoria Bridge over the St. Lawrence River at Montreal. It was erected between 1854 and 1860 from the design of the noted English engineer, Robert Stephenson, but was so expensive that the type was never approved by American engineers. It lasted a number of years in spite of the fact that the rivet heads came off so often that it was necessary to keep a gang of riveters constantly upon it in order to renew defective rivets. Finally, the structure was removed and replaced by a more modern American type of bridge.

Suspension bridges also were used by American railroads during this period. In 1855 John A. Roebling completed a suspension bridge of 821 feet span over the Niagara River just below the Falls for the Grand

Trunk Railway. In it deep, open-webbed, wooden stiffening-trusses were adopted. It was double-decked, carrying a fifteen-foot clear wagon-way below. A suspension bridge of three spans had been built a few years before at Frankfort, Ky. Owing to the lack of rigidity in the suspension bridge, that type of structure has not found favor among American railroad engineers.

The construction of long-span railway truss bridges in America dates from the early sixties, when the railroads began to cross the Ohio River. Linville built the first at Steubenville, Ohio, in 1863 and 1864. It was of the double-intersection, Murphy-Whipple type with cast iron for the compression members, and had a channel span of 320 feet. The earlier long-span bridges were usually of that type, though the Fink truss was employed to some extent. Both of those types permitted of economic construction with large truss-depths and short panel-lengths, which explains their popularity for long-span structures. The Fink truss was decidedly inferior to the Murphy-Whipple in respect to the important attribute of rigidity. The longest Fink truss had a span of 306 feet and the longest Murphy-Whipple one of 518 feet.

In 1864 David Reeves, of the Phœnix Bridge Company, introduced the Phœnix column of wrought iron with cast-iron bearing blocks, and developed the use of the hydraulic upset end for eye-bars. The said column was a great factor in causing the substitution of wrought iron for cast iron in compression members of pin-connected bridges. It served a good purpose for several years, but has been finally relegated into oblivion by better and more scientifically built columns. Cast iron continued to be used more or less in bridge construction until the Ashtabula Bridge disaster in 1876, after which its employment in railroad structures was practically abandoned, excepting only in the before-mentioned bearing-blocks of Phœnix column bridges.

Wooden stringers were generally used in railroad bridges until about 1873, when iron ones began to supplant them. The latter, however, had been adopted by the New York Central in the early sixties. The facilities for field riveting increased greatly about this time, and this explains largely the more extended adoption of the metal floor-system.

During the seventies the Pratt, the Whipple, and the Warren or Triangular trusses became the favorite types, although several large Post-truss bridges were then built. For a short span the single intersection type was found preferable, and for long ones the double or triple intersection. During this period C. H. Parker introduced the plan of making the top chords of through trusses polygonal, thus effecting quite an economy in weight of metal for long spans; and this modification of the Pratt truss is often termed the "Parker Truss." The Baltimore truss was originated by the Pennsylvania Railroad Company in 1871 by subdividing the panels of the Pratt truss; and it was subsequently modified by making the top chords of the through bridges polygonal, in which form it is sometimes known as the "Pennsylvania" truss. Both of these forms, though, are more frequently termed the "Petit" truss. The Fink truss has not been built for many years, and since the advent of the subdivided panel the Murphy-Whipple truss has gradually passed out of use, George S. Morison being the last of the prominent American engineers to adhere to it; and even he was compelled to abandon it for the more modern Petit type when he engineered the Merchants' Bridge over the Mississippi River at St. Louis. Today nearly all trusses of ordinary span length are being designed of the Pratt or Petit type, but occasionally the Triangular with secondary verticals is employed. The longest simple truss spans yet constructed are those of the Free Bridge (Fig. 1*g*) over the Mississippi River at St. Louis, where there are three Petit truss spans of 668 feet each; but there are still longer ones under construction and in contemplation, one of 723 feet for the proposed bridge over the Ohio River at Metropolis (Fig. 1*h*), and two of 775 feet each for the proposed crossing of the same river by the Chesapeake and Ohio Railway at Sciotoville (Fig. 1*i*). In the latter bridge the two trusses will be continuous. Continuous spans have been employed in only one structure of importance built in America thus far, viz., the Lachine Bridge over the St. Lawrence River near Montreal. It was constructed in 1885–7 from the design of the late C. Shaler Smith, the most prominent and progressive bridge engineer of his time. Owing to the great increase in live loading, it has lately been taken down and replaced by a double-track structure of simple spans.

The construction of real cantilever bridges in America began in 1876, when C. Shaler Smith built the Kentucky River Bridge at Dixville, Ky. He had originally intended to leave the chords continuous, and had assumed fixed points of contraflexure at what were later made the ends of the suspended span; but Mr. Bouscaren, Chief Engineer of the Cincinnati Southern Railway and himself a high authority on bridgework, refused to permit this, and Mr. Smith then arranged to cut the chords at these points after the erection was completed. The Eads Bridge over the Mississippi River at St. Louis was erected in 1873 by the cantilever method, the spans being arches without hinges. This plan of erection had been suggested as early as 1846 by Robert Stephenson, and several American engineers had proposed similar designing previous to 1876. The Solid Lever Bridge Company of Boston erected several combined arch and cantilever bridges of short span in New England and New Brunswick about 1868, some of which were used for railway traffic. The first one had solid cantilever arms of timber, but the later ones were open-webbed and of wrought iron. Since the building of the Kentucky River Bridge, the construction of cantilever structures has gradually extended. The longest existing spans of that type are still European, being the two of 1,710 feet each in the Forth Bridge, Scotland; but this limit is soon to be exceeded by the 1,800-foot span of the Quebec Bridge, which is now under construction.

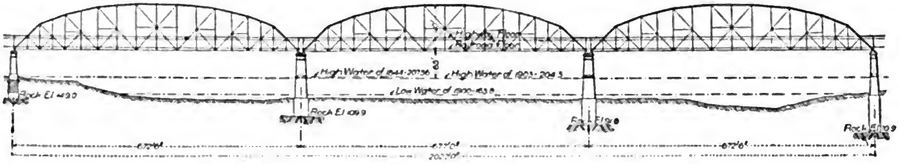

FIG. 1g. Free Bridge over the Mississippi River at St. Louis, Mo.

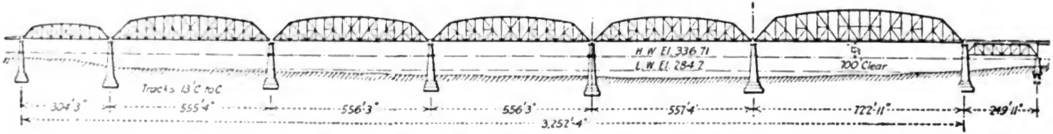

FIG. 1h. Proposed Bridge over the Ohio River at Metropolis, Ill.

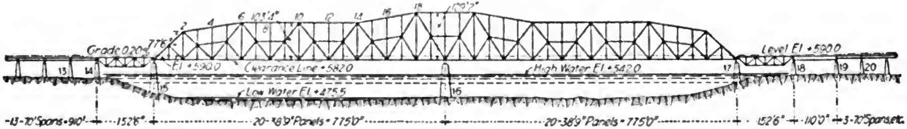

FIG. 1i. Proposed Bridge over the Ohio River at Sciotoville, O.

The metal arch has not been used as extensively in the United States as in Europe. The low banks and alluvial soils of so much of North America do not favor the arch type, which, generally speaking, is suitable only for comparatively high crossings and rock or other very solid foundations. Thomas Paine's attempt to introduce cast iron into American bridge practice has already been mentioned. In 1787 he advocated an arch bridge of that metal of 400 feet span at Philadelphia, but the project was not carried out. The first really important metal arch bridge in this country was the before-mentioned Eads Bridge at St. Louis, erected in 1873. It has cast, chrome-steel, hingeless ribs, the first application of this metal to bridgework in America; and, although it must be considerably overloaded, it ranks today as one of the finest bridges in the country. Since then a number of arch bridges have been built here—generally of the two-hinged or the three-hinged types. The largest arch span in the world is the one now under construction over Hell Gate at New York City, the span-length being 977 feet. It is of the two-hinged, braced-rib type. The spandrel-braced arch with two or three hinges has frequently been used, the most important existing bridge of the kind being that across the Niagara River near the Falls, designed and engineered by the late L. L. Buck.

In later years suspension bridges have been an important type for long-span structures, although not used for steam-railroad purposes. Three of the great bridges over the East River, viz., the Brooklyn, the Williamsburg, and the Manhattan, are of that type. Designs for suspension bridges over the North River at New York City have been made several times, notably in recent years by Lindenthal for an opening of 3,100 feet, and by Hodge with one of 2,880 feet. The theory of the stiffening-truss has been greatly improved in recent years, and the flexibility so noticeable in the earlier designs has largely disappeared in consequence.

The first wooden railway trestle was constructed on the Philadelphia and Reading Railway in 1840, as has been previously noted; and the boldest example of that type among early structures was the Portage Viaduct, designed by Silas Seymour in 1851. It was 800 feet long and 234 feet high. The first iron railway trestle was built in 1853 on the Baltimore and Ohio Railway, to which reference has already been made; and the first one of modern type was the Bullock Pen Viaduct, designed by Smith, Latrobe & Co. in 1867 for the Cincinnati and Louisville Short-line. To C. Shaler Smith in 1870 are probably due the conception and introduction of the braced tower, now so characteristic of American design; although T. C. Clarke claimed to have employed it in 1869 in his design for the approach to the proposed Blackwell's Island Bridge.

The gradual replacement of wood and cast iron by wrought iron has already been discussed. The first important use of steel for bridgework in the United States was in 1869 for the St. Louis Bridge, which was mentioned earlier. Its employment for eye-bars developed next, then flat plates suitable for floor-beam webs became procurable. The first American bridge of any consequence in which steel was used exclusively was the Glasgow Bridge over the Missouri River on the line of the Chicago and Alton Railway, built in 1879. The production of steel increased steadily, and in 1890 all the usual structural shapes could be procured at the same prices as for wrought iron; and by 1895 its adoption for bridges was practically universal, and the production of wrought iron in large quantities was a thing of the past.

Within the last decade investigations have been made concerning the employment of alloy steels for bridge superstructures, the result being that it has been found that nickel steel is a perfectly satisfactory material for bridge building and that its employment is in the line of economy, especially for long spans. It has been used in several large structures, notably the Blackwell's Island and Manhattan Bridges of New York City, the Free Bridge of St. Louis, the Fratt Bridge of Kansas City, and the new Quebec Bridge. Steel high in silicon is to be employed (for the compression members only) in the proposed Metropolis Bridge, before mentioned; and Mayarí steel, a low-grade natural alloy with nickel and chromium, has been adopted for the new Memphis Bridge. In the Hell Gate Arch Bridge high carbon steel was chosen because of the great prices asked by the bridge manufacturers for alloy steels. The future development in long-span bridge construction most assuredly will be determined largely along the lines of alloy steel manufacture.

Stone arch bridges have played a very small part in bridge evolution in America; but stone and brick were for many years the principal materials for substructure. The production of natural cement in this country began in 1818, and that material was much employed for concrete until the eighties, when Portland cement came into vogue and soon replaced it entirely. Concrete has almost totally ousted stone masonry from bridge construction.

Reinforced concrete was introduced into America about 1874 by Ransome; and in the following year W. E. Ward built a house of this material. In 1877 Hyatt published the results of tests which had been performed for him by David Kirkaldy; and shortly thereafter considerable construction work was done on the Pacific coast by Jackson, Percy, and Ransome. The latter patented the first deformed bar in 1884. The first application of reinforced concrete to bridge construction was in the early nineties. Within the next few years a large number of such structures were built, largely of the Melan arch type, von Emperger and Thacher being pioneers in this work. The past decade has witnessed its application to many other forms of bridge construction. For city bridges of short span its use is becoming almost universal, and it is being adopted extensively for short-span railway bridges and trestles. It also finds a very wide application in the construction of retaining walls and abutments, and of the floors of steel bridges.

American methods of bridge designing during the last six decades have certainly passed through a remarkable evolution; for, as previously noted, before 1847 all designing was purely empirical and in the hands of carpenters; during the fifties the knowledge of the methods of stress calculation spread, iron largely replaced wood as the material of bridge construction, and bridge designing came almost entirely into the hands of the railroad engineers, much of the manufacture being done by their forces in their own shops; but in the next decade some of the leading bridge engineers of the railroad companies started private shops and eventually secured most of the work, and soon the designing also came into the hands of the new bridge companies, most of them being especially interested in some particular type of construction on which they frequently held patents.

The age of keen competition then began; and while progress certainly was great, the tendency to skimp was still greater. As most of the railroad companies supplied simply a profile of the crossing, the loading, and possibly the span lengths, and asked for lump-sum bids, and then did not check the adopted plans or inspect the construction, there were every opportunity and temptation for the manufacturing companies to do poor work and "skin" their structures. But during the seventies slowly there came a change. Specifications for bridge work then began to appear—first by Clarke, Reeves & Co., in 1871, then by George S. Morison for the Erie Railroad in 1873 (probably the first printed bridge specifications ever adopted by any American railroad), then in 1875 by L. F. G. Bouscaren for the Cincinnati Southern Railway, the first road to specify wheel-load concentrations. Morison required successful bidders to submit stress sheets and plans for approval before starting work, and later began the inspection of materials and workmanship; and Bouscaren soon adopted the same policy. Inspection, though, was first inaugurated on the Eads Bridge in 1869. In 1877 Charles Hilton prepared bridge specifications for the Lake Shore and Michigan Southern Railway, and in the same year C. Shaler Smith issued some for the Chicago, Milwaukee, and St. Paul Railway; and in 1878 Theodore Cooper got out a set for the Erie Railroad. This was the first of Cooper's bridge specifications, which, as revised from time to time, have been very widely used. Other railroad companies soon began to develop bridge engineering forces, and by 1876 some of the most competent of the bridge engineers were entering private practice as consulting engineers. The lack of rigidity and lateral stability of most of the structures built previously was rapidly becoming apparent, especially to the engineers who were acting in the interests of the railroad companies and other clients; and the need for the making of complete designs by competent engineers, rather than by the cheap draftsmen of competing bridge companies, began to be realized. It became evident that while the old competitive system of lump-sum bids by the various bridge companies on their own plans had played its part in developing economic design in America, for best practice it had become a thing of the past. As C. C. Schneider has stated, "The manufacturer should confine himself to his legitimate field of manufacturing steel at so much a pound." The best practice today consists in having the complete detailed designs and the specifications prepared by a bridge specialist, either regularly employed by the purchaser or retained by him for the special work, and then to ask for unit-price bids thereon from first-class contractors only. That practice prevails today almost universally in railroad work, and is rapidly coming into vogue for the better class of city structures; but the older practice of competitive, lump-sum bids still holds in small cities and towns and in the country districts. The establishment of State Highway Commissions in several of the States, such as Iowa and Illinois, with competent engineers in charge, is doing much to correct the evils of highway-bridge letting.

Perhaps the most important features of the development in bridge designing of late years have been the great advance in the science of detailing and proportioning, and a growing recognition of the fact that a bridge should be very effectively braced, and should be thoroughly rigid in all of its parts. Until the early eighties very little attention was given to the detailing, the stresses and sections for main members being calculated carefully by the engineer, but the still more important task of designing the connections for the said main members being turned over to cheap draftsmen. The detailing that used to go into bridges thirty or forty years ago would make the hair of a modern bridge engineer rise with horror! How such crudities could have been permitted is almost beyond comprehension today; for the veriest tyro can now see that the details of those old structures were incapable of carrying more than a small percentage of the stresses for which the main members they connected were proportioned. Furthermore, the said earlier structures were very poorly braced, so that they racked to pieces rapidly. As the designing and detailing began to come into the hands of experienced engineers, a movement to improve details and bracing began, but it progressed with painful slowness at first, for during the eighties bridge detailing was about as crude and unscientific as can be imagined. Toward the end of the eighties pin-connections for short spans commenced to be dropped out of the best practice, the flimsiness of the web members in many of the multiple-intersection structures began to make itself evident, and the desirability of single-intersection systems from this standpoint was realized. The great value of plate-girders for short spans was recognized, and their employment increased rapidly. From that time onward the evolution has been steady, the leaders therein being the independent bridge specialists and the bridge engineers of the various railroad systems. The American Railway Engineering Association, organized in 1899 as the American Railway Engineering and Maintenance of Way Association, by the wide-spread use of its specifications and by its many and careful prepared committee reports, has raised materially the standard of railway-bridge designing; and the American Society for Testing Materials has been the means of effecting a number of improvements in the quality of the various materials employed in bridge building. But by far the most effective cause of the substantial and fundamental advance in the science of bridge engineering has been the publication by the American Society of Civil Engineers of several papers on important bridge subjects written by leading specialists and discussed widely by members of the society connected in various ways with bridge designing, building, and operation. Some of these papers put a stop to many glaring faults of design and construction, and others offered suggestions for future development which the profession has followed. As a result of this evolution during the past twenty-five or thirty years, a first-class American bridge of today is a very rigid structure, effectively braced against all possible lateral forces, and carefully proportioned in all of its parts.

The question of riveted versus pin-connected trusses has always been a mooted one among American engineers. While European practice has always favored the riveted type, early American practice endorsed the use of pins, the New York Central Railroad being a conspicuous exception, as it has employed riveted construction altogether. The introduction of the pneumatic riveter served to remove many of the objections to field-riveted connections; and of late years the riveted type, as improved and more scientifically designed by American engineers, has found much favor in this country on account of its superior rigidity, specially because much of the former economy of pin-connections has disappeared as the necessity for making many of the tension members stiff has been realized. Whereas thirty years ago many American engineers would have used pin-connected spans of 100 feet, today most of hem advocate riveted ones for openings up to 250 or 300 feet—or even more. Three simple-truss, riveted spans, each of 425 feet, of the heaviest kind of construction, were used in 1909–1912 in building the author's Pratt Bridge over the Missouri River at Kansas City. (See Figs. 31*d* and 1*e*.) The Sciotoville Bridge (Fig. 1*i*) with its 775-foot spans and the Hell Gate arch (Fig. 26*g*) with its 977-foot span will have riveted con-ections throughout, and they will be used extensively in the Quebec antilever bridge. The extremely heavy riveted connections required for hese structures have led to many advances in shop practice, such as the eaming or drilling of field connections with the parts temporarily assem-ed, the use of very large and long rivets (such as those of 1¼ inches ameter and 10-inch grip in the Hell Gate arch), and the adoption of apered rivets where very long grips are necessary.

The evolution in erection methods has also been marked. The intro-uction of the cantilever and the cantilever method of erection for arches d simple spans many years ago made feasible the construction of bridges places where the employment of falsework either interfered with navi-

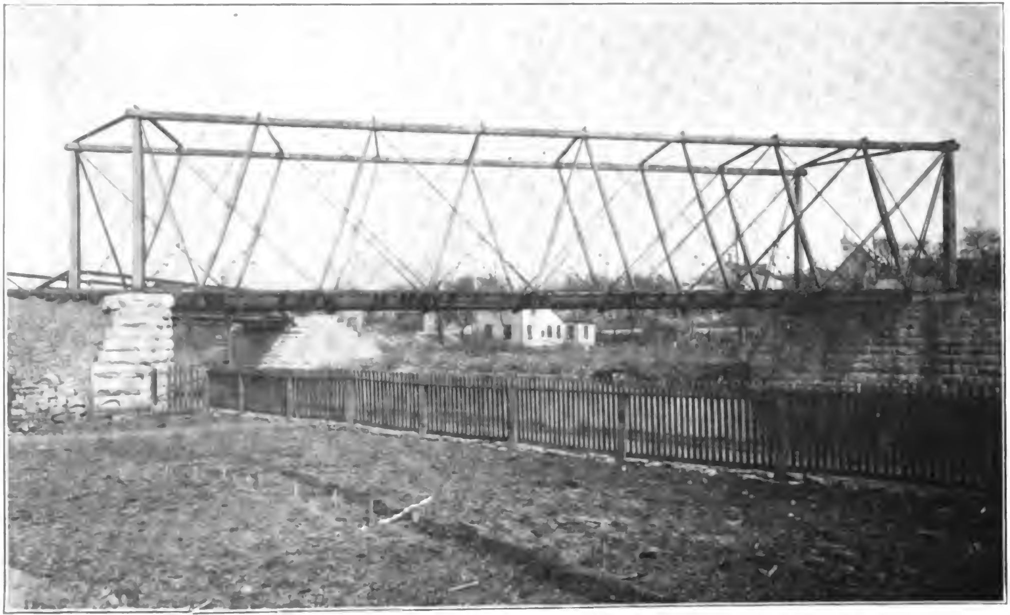

FIG. 13. Chicago and Alton Railroad Bridge over the Illinois and Mississippi Canal at Lockport, Ill.

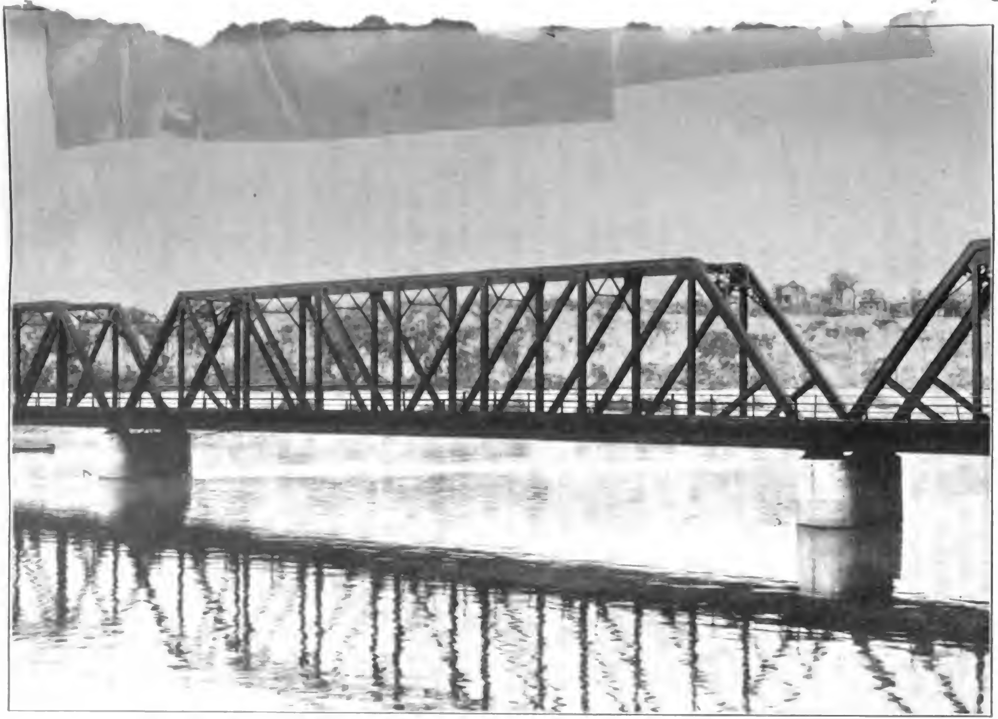

Fig. 1*k*. Kansas City Southern Railway Bridge over the Kaw River at Kansas City, Mo.

gation or was extremely hazardous or entirely impracticable; and the later method of erecting a span complete on barges and then floating it into position solved the same problem in another way. The refusal of many railway companies to permit their traffic to be interfered with has led to the introduction of many new erection methods. A most noteworthy feature has been the replacement of the traveller by the derrick car for the erection of structures of ordinary size, resulting in the cheapening and accelerating of the field work.

The part played in the evolution of bridge designing by changes in methods of transportation and by the increase in commerce should not pass unnoticed. Necessity has been the mother of invention in this as in other things. Primitive man needed but a slight structure to sustain himself or his horse, and his commerce amounted to almost nothing. As wheeled vehicles appeared, wider and stronger structures were required; but the loads still were comparatively light, and the commerce being chiefly water-borne, bridges were more a convenience than a necessity. In 1784, when the mail-coach service of England was improved and extended, the immediate result was the passage of some three hundred acts for the construction of roads and bridges within the next decade. But it was not until the railroads began to push out in all directions and to carry a heavy traffic that the real development began. Bridges then became truly a necessity, and their design progressed rapidly. Furthermore, the engines and trains increased constantly in weight, so that a bridge became seriously overloaded in the short space of ten or fifteen years. This progress in the growth of railway rolling stock and the loads carried therein caused a steady increase in the capacities of railroad bridges. In the early days of railroading the Baltimore and Ohio was using the four-wheeled, grasshopper type of engine weighing only 22,000 lbs., and its loaded cars weighed not to exceed one ton per lineal foot of track, while today there are in service Mallet locomotives weighing 61 000 lbs. without the tender, or 840,000 lbs. including it; and there are on record car-loads of 7,300 lbs. per lineal foot. The engine axle loadings have increased from 11,000 lbs. in 1835 to 65,000 lbs. or more today, and the end is not yet. Of late years the increase of street-railway and interurban car traffic has required that important city bridges have the same elements of strength as railway bridges; and quite lately the rapid increase in the size and use of motor trucks and the improvements in the roads of the country districts have made it imperative that bridges in those locations be designed for heavy loads.

For the purpose of showing the difference between early and modern railroad bridges the author has concluded to insert Figs. 1*j* and 1*k*. The former shows a Post-truss, single-track, railway bridge at Lockport, Ill., over the Illinois and Michigan Canal, built in the sixties and in operation a few years ago (and possibly today) on the line of the Chicago and Alton Railroad. The posts are of cast iron and hollow. The latter in contradistinction represents a single-track span of the author's built about 1900 in the bridge across the Kaw River at Kansas City, Mo., on the line of the Kansas City Southern Railway.

The evolution of movable bridges is discussed incidentally in Chapter XXVIII, which treats of "Movable Bridges in General," and in the three succeeding chapters, which deal at length with the three most prominent types of moving spans.

From the foregoing it is seen that the principal factors governing the evolution of bridge building have been—

Available materials,
Advent of new materials,
Pre-existing forms,
Extent of knowledge of the laws of mechanics and of the properties of materials,
Shop practice and facilities,
Transportation needs, and
Erection methods.

The requirements of transportation will undoubtedly make further calls upon the structural engineer and the bridge constructor; because heavier loads, greater density of traffic, and the demand for better connections between more widely separated termini will continue to crowd hard upon existing limitations and cause an increased effort to remove such restrictions. The desired result may be reached by the use of some new alloy of steel, by an increase in our theoretical knowledge, by an improvement in shop practice and equipment, or by a further development of erection methods; and possibly all these factors may enter into the further evolution of bridge building. The limits in that art have certainly not yet been reached—far from it!—for the era of long-span bridges has only just begun; and he would, indeed, be a bold prognosticator who would dare to set a bound to the possibilities of attainment of the next generation or two of American bridge engineering specialists.

CHAPTER II

THE BRIDGE SPECIALIST

Specialization in all lines of activity is the order of the day, and the movement has extended to the professions as well as to manufacturing and general business. A century or two ago it was possible for a learned man to accumulate a large share of all that was valuable of scientific knowledge, then later such a man had to content himself with knowing everything worth while in a single line of learning, but today he must devote his attention and energies to a small subdivision of one of these lines.

In law, besides the two general classes of trial lawyers and consulting lawyers, there are specialists in corporation work, in criminal cases, in patents, in shipping, in pleading, in railroading, in insurance, in real estate, in conveyancing, and in personal injuries.

In medicine there are specialists not only for many single diseases but also for the exclusive treatment of certain different parts of the body, surgeons who do not prescribe medicine, physicians who never perform a surgical operation, doctors who diagnose only, and mental specialists, besides others who are often considered outside the pale of the true medical profession, such as osteopaths and mental science healers.

In engineering there are also specialists, and their name is fast becoming legion. Years ago there were but two divisions in the profession, viz., civil and military; then the former became divided into civil, mechanical, electrical, architectural, chemical, metallurgical, mining, and marine engineering; but today each of these divisions is further divided. For instance, the modern term "civil engineering" covers bridge engineering, hydraulic engineering, municipal engineering, sanitary engineering, railroad engineering, highway engineering, and landscape engineering. Even some of these specialties are becoming subdivided, for there are hydraulic engineers who devote themselves exclusively to water-works, others who confine their attention to river improvement, and others who specialize in harbors. Again, among railroad engineers there are some who do nothing but surveying, others who attend solely to construction, and still others who are entirely in the line of operation or maintenance.

Bridge engineering has been recognized as a specialty for more than a quarter of a century. It was probably the first branch of civil engineering to segregate from the general professional practice; and, in consequence, it has become more highly developed as a specialty than any other line of engineering work. Notwithstanding this, the general public is far from being convinced that bridge engineering is a specialty, and that bridges should be designed and their construction supervised by trained specialists only. Many high railway and city officials and promoters of important projects appear to think that any kind of an engineer can design and supervise the building of their bridges, or that the designing can be done by the manufacturer of the superstructure metal and the field supervision by any low-salaried engineer or surveyor. To this idea are due the facts that there are so many bridge failures throughout the country, that the life of an iron or steel bridge is so short, and that one of the greatest of all railroad expenses is the renewal of metal bridges. Railroad officials and the public generally, on account of these frequent renewals, have reached the conclusion that a steel bridge has to be replaced regularly every few years, and hence there is no need to go to extra expense in its design or construction in order to obtain problematical improvements. Such an idea is entirely erroneous, because the failure and wearing out of the superstructure of metal bridges are due primarily and almost exclusively to faulty designing, especially in the details. A modern steel bridge carefully designed by a first-class bridge engineer, constructed under proper supervision and thoroughly painted from time to time, will last for an indefinitely long period, even if it be materially overloaded.

One of the most serious difficulties that a bridge specialist has to encounter is the necessity for convincing possible clients that his services have a real money value and are not in the nature of a luxury. Eloquence may enable him to do so; but, if not, time surely will; because the user of a badly designed bridge, if he live the ordinary span of life of a business man, will inevitably learn that he has made a serious mistake in entrusting to incompetent or interested parties the designing and the supervision of the manufacture and erection of his bridges. A good proof of the value of a bridge specialist's services is the fact that whenever a railroad official has once employed a competent consulting bridge engineer, he will rarely ever build any more important bridges without retaining an expert to do the designing and to attend to the supervision.

In the entire history of engineering construction no greater mistake can be found than the entrusting of bridge designing to the superstructure manufacturers; for naturally, with a few notable exceptions, their main object is to make all the money they can out of the contract, irrespective of the interests of the purchaser. Their aim is to push the metal work through the shops as quickly and cheaply as practicable in order to turn out in any given time the greatest possible tonnage of metal. To do this the detailing is simplified in every way, and the number of rivets is reduced to an absolute minimum, not only for shop work but also for field work, as the manufacturer, especially on large bridges, generally contracts for the erection. Each rivet omitted means so many cents saved, but usually the less the number of rivets in any connection the weaker the detail, hence what is gained by the manufacturer and erector is lost by the owner. It needs only a very small reduction in the required number of rivets for a connection so to reduce its strength as to permit of its wearing out quickly; consequently a few cents saved in a detail may not only shorten materially the life of a structure but also cause a frightful disaster with great loss of life. Again, the manufacturers' engineers generally design bridges solely for the known stresses as shown on the stress sheet, and usually neglect to consider the effect of secondary stresses and that of vibration or impact, while the expert bridge specialist always gives these important matters due consideration.

The Quebec Bridge failure is a glaring example of the evil effects of leaving the designing to the bridge manufacturer; for while in that case there was nominally a consulting engineer, he did not prepare either stress sheets or details, but entrusted this vitally important work to the manufacturers.

In connection with this great disaster an important question pertinent to the subject of this chapter was raised by an erroneous statement in the testimony of one of the witnesses, and this statement was indirectly endorsed editorially by *Engineering News*. Later in the columns of that paper the author challenged it, and as the matter is of great importance to both the engineering profession and its clients, his letter is here reproduced. It reads thus:

"In connection with the investigation of the Quebec Bridge disaster a point has been raised which, in my opinion, is likely to produce an erroneous impression on the minds of your readers. I, therefore, beg permission to call attention to the matter and to give you my views thereon for publication.

"From p. 475 in your issue of October 31, I quote the following:

"'*Q*. Do you consider that it is wise practice when building a bridge of novel character and unprecedented dimensions to place the design of the structure and of the methods of erection in the hands of the mechanically trained staff of a contracting company, and, if not, why was this practice allowed in this case?

"'*A*. In answer to this question, it is the general practice in America to have the mechanically trained staff of contracting companies prepare the working plans. As a rule, no engineer could afford to maintain a staff of such character and no corporation would listen to a fee that would cover any such expense.'

"And from your editorial on p. 587 in the issue of November 28, I quote this statement:

"'The normal and proper way of constructing any great engineering work which is done by contract is for two engineering organizations to work together. The one representing the contractor and the other representing the purchaser can check each other's errors, can study the work from different points of view, and can produce a better and more economical result working in co-operation than either could working alone.'

"If these statements are not contradicted, the general public will be led to believe that American bridge specialists are incapable of designing great bridges, that they have to fall back on the superior (?) knowledge of the bridge companies, and that their sole function is to check the numerical work (and perhaps also the honesty) of the manufacturers' designers.

"Unfortunately, the old and still too prevalent custom of railroad companies of having stress sheets prepared by their own engineers and of leaving the detailing to the bridge manufacturers lends some credence to this notion; but I beg to state most emphatically that any consulting bridge engineer who is truly worthy of that distinguished appellation is not only capable of preparing complete designs for bridges of all kinds, but also makes an exclusive practice of so doing.

"The consulting engineer who prepares specifications, or specifications and stress sheets, only, and submits them to bridge manufacturers for the detailing, is shirking his work and is not earning his pay. Once in a while some railroad company, as a matter of supposed economy, insists that its designing must be done in this manner; but the result is almost invariably unsatisfactory in that, if the engineer fail not in his full duty, he will do far more work than he is paid for, and in that the resulting structure is inferior to what the engineer would himself have designed, because the outcome as regards the detailing is invariably a compromise. On a few occasions I have been placed in this predicament, and the result has always been unsatisfactory. As a matter of fact, little or no economy results; for when the contractor does the detailing, the cost thereof must be borne by the purchaser just as truly as though the consulting engineer did the work and was paid directly for it; because this item is hidden in the total cost of the structure. Such economy as the practice entails springs from the fact that the contractor employs cheaper and less competent men to do the work than the consulting engineer does.

"That the railroad companies of this country are beginning to understand this matter is shown by the fact that a number of the principal systems have established bridge departments of their own for the purpose of preparing complete detail plans for all the new bridges required, as well as to supervise the maintenance of old structures. Again, many other railroad systems employ independent consulting engineers to do all their bridge designing and to supervise the construction of all important structures.

"To the practice of letting the superstructure manufacturers prepare the plans of bridges is due the fact that so many railroad structures wear out and have to be renewed. Such structures fail invariably in the details. An old bridge designed by a manufacturing company ordinarily reaches the danger limit when, according to the best modern specifications for designing, it is overstressed fifty (50) per cent, or in some cases even less; while a really scientifically detailed bridge will be perfectly safe under much greater overloads.

"In defence of the few American bridge specialists who can properly claim to be entitled 'consulting bridge engineers,' and who are entirely disassociated from the manufacturers, my firm being included, I desire to state that we have offices so organized that the entire designing in every detail of a structure as large and important as the Quebec Bridge, or even larger, would be done by us without any aid from the contractors. It is true that we might have to consult them occasionally as to the ability of their shops to do certain work in a certain way; but most assuredly we should never have to ask their assistance in making the plans.

"A true bridge expert is an engineer who is thoroughly posted in every detail of designing and construction, and who has had ample experience not only in the designing office, but also in the rolling mills, the bridge shops, the testing room, and the field. It is in the defence of such engineers that I am sending you this letter."

It is not only in large structures that the services of the bridge specialist are required. Small ones also need careful, scientific designing, close inspection in rolling mills and shops, and thorough supervision in the field, if the purchaser is to obtain the full worth of his money. Accidents and failures are by no means confined to large structures—in fact, far more of them happen to small bridges than to large ones, not only because there are more of the former than of the latter, but also because, as a rule, more care is given to the designing, manufacture, and erection of large and expensive structures than is usual in the case of small and cheap ones. Bridges as insignificant as rolled I-beam spans often require the trained judgment of the bridge specialist in their designing. This was shown once in the author's practice, when one of his clients, in order to save a small fee for designing, had a little ten-foot span designed by his railroad engineer, with the result that on account of his forgetting the heavy, single-axle concentration from one of the passenger locomotives, some ten or twelve of these little structures had to be strengthened by doubling the number of lines of I-beams.

In the days when it was customary for railroad companies to call for competitive bids and to let their bridges by the lump sum to the lowest bidder (and, alas, this pernicious custom has not yet quite passed out of existence) the evil effects of doing bridgework without competent engineers was more marked than it is today, as it is now usual to purchase railroad bridges by the pound either delivered on cars or erected. Still today the bridge specialist is needed not only to secure proper designs but also to protect his clients' interests against overcharge and fraud; because when bridges are paid for by the pound and when the designing is left to the manufacturer there is a great temptation to put in much unnecessary metal, generally in the main members instead of in the details where it might do good by stiffening the structure and increasing its resistance to impact. An amusing incident once occurred in the author's experience that confirms the last statement and illustrates clearly the necessity for disinterested engineering supervision.

An engineer was engaged in compiling the records of weights of metal in bridges designed by one of the largest American bridge companies, and could not derive satisfactory curves for his diagrams because of the great variation in the weights per foot of structure. One day in despair he went to the Chief Engineer with a glaring case of variation and said: "How do you account for this?—here are two bridges of the same span, designed for the same live load and under the same specifications, and yet their weights of metal per lineal foot vary twenty-five per cent!" The reply was: "My dear fellow, that is dead easy, one was built for a lump sum and the other for a pound price."

Even if the bridge companies were always perfectly honest in designing bridges, they could not obtain the best possible structures, because their designers spend their entire lives in the offices of the company and never see in the field how their bridges act under load; while, on the other hand, both the independent consulting bridge engineers and the salaried bridge engineers of railways are continually inspecting defective structures, learning their weak points, and evolving methods of improving designs for new bridges. In this particular the consulting engineer has a decided advantage over his salaried brother, for the former's experience is with many roads, while the latter's is generally confined to but one.

There are very few chief engineers of railways who are at all thoroughly posted about bridgework; and the higher such men stand professionally the more ready they usually are to confess their ignorance and to call in specialists to aid them in designing and building their structures. It is only the small, narrow-gauge man who claims that he knows all about everything; and it stands to reason that, if a railroad engineer is really an expert on bridges, he is almost sure to be deficient in general knowledge of many important branches of engineering connected with railroad work; for nowadays life is too short to cover the entire field of professional knowledge and experience in such a broad subject as railroading. When a railroad engineer claims that he is "a pretty good bridge engineer himself," as he sometimes does, it is safe to put him down as a "jack of all trades and a master of none."

Except in the case of very large and wealthy railway companies which can afford to pay big salaries for engineering work, it is better for a railroad company to retain some bridge specialist or some firm of consulting bridge engineers to attend to all the engineering of its bridgework rather than to establish a corps of bridge engineers of its own. The reasons for this are as follows:

First. The work will be better done by a trained specialist than by a mediocre man working on a salary.

Second. In case of accident to a bridge involving loss of life, the railroad company will escape more easily both public censure and heavy damages if it can show that it did its utmost to avoid such trouble by employing specialists eminent in their profession to engineer its structures.

Third. It is generally less expensive in the end to pay specialists the regular standard percentage fees than to maintain constantly on salaries a regular bridge engineering force, because when the specialists are not working for the company they are not paid, while the salaries go on month after month, year after year, regardless of whether the men are busy or idle; and it is not practicable to discharge salaried bridge engineers when work runs short and to pick up others when it is resumed, if one expects to have the work done satisfactorily and effectively.

Too often railroad companies deem that any cheap engineer who has had a few years of experience in bridge drafting will suffice for their regular bridge engineer; and while they do not always learn it quickly, they are continually paying for his mistakes and lack of wide experience. A glaring case of this kind once came to the author's knowledge, in which the chief draftsman of a bridge manufacturing company was secured for a small salary to act as bridge engineer for an important railroad system. Some of the designs that he prepared were extravagant and others were absurd. For instance, in a long, high, deck bridge he widened at great expense the piers so as to reduce their height and surmounted them with braced steel towers; and he evolved a hybrid truss, partly riveted and partly pin-connected, the pins being located on the axial lines for the eye-bar diagonals but eccentric to those of the chords. The best shop work possible cannot ensure that for such construction the pin centres shall be in exact position, and as the eye-bar diagonals were not provided with means for adjustment, the result is certain to be that some of the diagonals will be loose and that others will be overstressed. The errors of this cheap engineer will eventually cost his company many times the amount of the fees that it would have had to pay competent specialists for doing the designing.

When a railroad company employs a bridge specialist, it should have complete confidence in him and should leave all the details of the bridge work entirely in his hands, being guided by his advice and counsel in all matters relating to the structures for the line. Any interference with him in his duties will breed trouble and expense; and when the president or general manager thinks that he knows more about bridges than does the specialist, it is time to make a change in the manner of handling the bridgework of the road; for no self-respecting engineer should permit unwarranted interference with his duties by any one, even the president of the railway.

Examples of the ill effects of such interference have occurred in the author's practice, among which may be mentioned the following:

On some foreign work the president of the road requested the resident engineer, who represented the bridge specialists, to change the specifications for building certain small piers and abutments so as to permit therein the use of lime instead of Portland cement. The resident engineer very properly objected and reported the matter to his principals, who refused in writing to make the change; then, when the president insisted upon the modification, they arranged to have the supervision of the construction of all the said piers and abutments taken out of their jurisdiction and transferred to that of the chief engineer of the railroad. The result was that over one hundred thousand dollars' worth of substructure had to be taken out and replaced because it was not strong enough to carry even the dead load of the spans.

In another case a design for a small bridge did not please the chief engineer because of *alleged* lack of economy due to excessive length of superstructure, and he persuaded the president to order it shortened. The original design involved a cheap buried pier at each end to support a 50-foot, deck, plate-girder span, the middle span being of the same type but 70 feet long, and the intermediate supports being steel bents on little concrete pedestals. The alternative design, which in spite of an emphatic, written protest, was adopted, consisted of a 125-foot deck, open-webbed, riveted truss span resting on two high concrete abutments. The total cost of the structure was increased twenty-five per cent by the change.

In another case the bridge engineers were compelled to use an inferior cement because its manufacturer would be an important patron of the railroad; and when they tendered their resignation they were begged to finish the work on the understanding that they were not to be held responsible for the efficiency of the substructure. While the concrete thereof hardened after a long time, the piers are certainly not as good as they would have been had first-class cement been employed.

In another case the superintendent of bridges of a railroad, to whom the bridge specialists reported, insisted upon the resident engineer raising the base of a concrete abutment, which rested on wooden piles, two feet above low-water elevation, simply because the excavation was endangering his old timber trestle. The resident engineer was weak enough to comply, instead of reporting the matter to his principals, who would certainly have refused to permit the change; and the result is a pile foundation that some day may fail because the piles are exposed to the air. However, as they are surrounded by earth up to the concrete and are likely to remain so, it is probable that the evil day is far distant. It would have been much better, though, to spend a few dollars on reinforcing the old trestle rather than subject the abutment of an otherwise truly first-class bridge to the possibility of failure even in the distant future.

The ideal bridge specialist is an engineer who has had a broad, liberal, general education, who is a graduate of some technical school of high standing, who has had a varied experience in several other lines of engineering than bridges, who has a practical knowledge of all the work in the rolling mills and bridge shops, who is fairly expert in testing materials, who is versed in the commercial as well as in the professional features of bridge building, who has served for several years in both office and field under some competent consulting bridge engineer before starting in practice for himself, who has developed sound judgment from being connected with the handling of large enterprises, who is both honest toward his clients and fair toward his contractors, and who is not only eminently energetic himself but also capable of making every man under him exert his best efforts on the work. The bridge specialist should have some knowledge of the law of contracts, should be expert in the preparation of specifications, contracts, estimates, reports, prospectuses, and similar papers, and should be such a master of his own language that his literary productions are models of clearness, vigor, thoroughness, completeness, and elegance of diction. He should ever be both a student and practitioner of true economy in design and construction, and he should pay special attention to rigidity as well as strength. He should give due attention to æsthetics in designing by beautifying in every practicable way the structures that he plans and builds. He should also study the action of old structures under passing loads not only to determine concerning their safety, but also to ascertain their weaknesses so as to avoid them in future designs. He should be systematic in all that he does, and he should make a practice of so recording and filing that everything of value connected with his work may be readily found. In dealing with contractors, while primarily protecting his clients' interests, his position should be judicial. In all business relations he should be tactful and polite, for otherwise he will either fail to accomplish desired results or will do so by the expenditure of much wasted energy and effort. While being careful and painstaking in every detail of his work, he should know how to transfer to others the burden of such details and to see that they give them the necessary attention. While conservative in guarding his clients' interests, he should ever be progressive; and, instead of being a worshipper of precedent, he should become an advocate and practitioner of legitimate innovation. And, finally, he should give to the engineering profession the benefit of his researches, discoveries, and accumulated knowledge by writing technical books, and by preparing papers for the engineering societies, instead of selfishly reserving such information for his own personal use and benefit.

The life of a bridge specialist is by no means easy, for like every one else he has his grievances; but he must learn to bear with those that are unavoidable and overcome the rest; and his governing motto should ever be "integrity, thoroughness, and progress."

CHAPTER III

ORDINARY MATERIALS OF BRIDGE CONSTRUCTION

In this chapter there will be discussed rather concisely each of the ordinary materials employed in the construction of bridges, beginning with those for superstructure, then passing to those for substructure, shore protection, etc. Alloy steels, however, the use of which is a new departure in bridge building, will be reserved for the next chapter.

ROLLED CARBON STEEL

Ordinary rolled steel used for bridge superstructures is divided into three classes, viz., soft, medium, and high, but the exact limits thereof are not accurately determined. Without laying oneself open to severe criticism, it may be stated that soft steel has an ultimate strength of from 50,000 lbs. to 60,000 lbs. per square inch; medium steel from 60,000 lbs. to 70,000 lbs. per square inch; and high steel from 70,000 lbs. to 80,000 lbs. per square inch.

Soft steel is mainly used for rivets and adjustable rods, and medium steel for most of the other parts of bridges. High steel in the past has occasionally been employed for eye-bars in bridges of long span, but of late it has been replaced by nickel steel. It is legitimate to make pins and expansion rollers of high steel; but it is hardly worth while, for their weight is such a small percentage of that of the whole superstructure that it would scarcely pay to use a special steel for their manufacture, unless it were really desirable to reduce the diameters of the pins or the sizes of the roller bases. High steel has sometimes been employed for the manufacture of built members, but, really, it is unfit for this purpose, as it is too brittle to withstand properly the various manipulations to which bridge metal is subjected in the shops.

The current practice of American steel manufacturers is to make but little, if any, distinction between the soft and the medium steels used for bridgework. They keep the ultimate strength of most of their product down to from 60,000 lbs. to 62,000 lbs. per square inch, their object in so doing being purely commercial. It costs no more to manufacture medium steel of strength between 60,000 lbs. and 70,000 lbs. per square inch and having an average of 66,000 lbs. than it does to manufacture a steel having an average strength of 4,000 lbs. or 5,000 lbs. per square inch less, excepting, perhaps, that the higher product is slightly more liable to rejections. As far as the manufacture of bridges is concerned, it really costs no more to use the true medium steel than it does to employ the compromise product of the manufacturers; and the structures built from the higher metal are in every way as good and reliable as those built from the lower, while they have five (5) or six (6) per cent greater strength. One of the manufacturer's principal objects in using the softer steel for bridges is to avoid reaming the rivet holes; but such avoidance for any kind of steel is not good practice. No matter how soft the metal may be, it should be reamed: primarily, so as to make the holes of the component pieces match properly; and, secondarily, so as to remove most of the metal that is injured during the process of punching. This question is of such importance that it is dealt with at length elsewhere in this treatise. The composition and qualities of rolled carbon steel of all kinds used in bridgework are treated fully in Chapter LXXIX.

In times past there have been many discussions concerning the relative merits of Bessemer, acid open-hearth, and basic open-hearth steels. The author has always opposed the use of Bessemer steel for bridgework on the ground that it is unreliable and subject to cracking, and today the stand he has taken is confirmed by the established practice of the best American bridge engineers, who unanimously bar Bessemer steel from their structures. For many years acid open-hearth steel was rightly considered superior to basic open-hearth steel in that it was more reliable, but the process of manufacture of the latter product has been so much improved that it is now superior to the former—in fact, it is today used almost exclusively in bridge construction.

Those interested in the designing and construction of metallic structures are often asked how much carbon there is in the various kinds of steel used therein, and generally the question remains unanswered because of inability to reply, the reason being that the amount of carbon is not specified by the bridge engineer, but is left to the discretion of the metal manufacturer. The author, of course, has had for many years a general idea of the amounts used; but, in order to speak on the subject authoritatively, a short time ago he asked one of the high officials of the Carnegie Steel Company to state the amounts to him, and he obtained in that manner the information given in the following table.

CARBON STEEL

Ultimate Strength in Pounds per Square Inch	Percentages of Carbon in the Steel
50,000	0.25 to 0.30
60,000	0.30 to 0.35
70,000	0.35 to 0.40
80,000	0.40 to 0.45
90,000	0.45 to 0.50

The amount of manganese in such carbon steels varies uniformly from 0.5 per cent in the soft steel to 0.7 per cent in the highest steel.

The ordinary metal sections employed in American bridge designing are as follows: plates, angles, I-beams, channels, flats, Z-bars, buckled plates, trough-sections, corrugated plates, H-sections, tees, and reinforcing bars.

Plates are rolled in width as great as eleven (11) feet, and in length up to seventy (70) feet for the narrowest and eighteen (18) feet for the widest sizes, the thickness being limited to two and a quarter (2¼) inches. Plates of even greater dimensions than these can sometimes be obtained by paying a special price for them. There are two kinds of plates used, viz., sheared plates and universal mill plates. The latter are limited in width to about four (4) feet.

Angles are rolled up to the limit of eight (8) inches by eight (8) inches and up to a thickness of an inch and an eighth (1⅛). For ordinary sections they can be obtained up to one hundred (100) feet and over in length; but for very heavy sections the limit is less. There is no hard and fast limit of length given by the manufacturers; and it is probable that for special cases great lengths for the heavy sections could be procured by paying a special pound price. It is not good policy, though, to order special sections or lengths for any rolled metal, because of the delays that are usually involved in the execution of such orders.

The ordinary limit of depth for I-beams is twenty-four (24) inches, but the Bethlehem Steel Company on its list of special sections has beams of 26, 28, and 30 inches depth. That Company at first experienced serious difficulty with its special sections; but it has been entirely overcome. This is fortunate, because these very deep beams are a great boon to bridge designers and builders.

With the exception of some eighteen (18) inch channels listed by the Cambria Steel Company, but not yet in general use, none of the sections deeper than fifteen (15) inches have been employed. There is an impression prevalent that the deeper sections warp badly in cooling. There is no limiting length for I-beams and channels set by the rolling mills, and the bridge designer generally finds no difficulty in procuring these sections in as great lengths as he desires.

Flats can be obtained up to any length and section needed in bridge designing. Z-bars are rolled up to six (6) inches depth only and to a thickness of seven-eighths (⅞) of an inch. As this type of section makes an excellent column, it is to be hoped that the American manufacturers will soon roll larger sizes.

Buckled plates are manufactured up to four (4) feet in width and in lengths up to about thirty (30) feet, the rise being limited to three and a half (3½) inches.

Trough plates when riveted together form troughs about six (6) inches deep with a distance of eight (8) inches between centres of adjoining sections.

Corrugated plates are rolled in width from eight (8) to twelve (12) inches, in thickness up to one-half (½) inch, and in rise from one and a half (1½) to two and three-quarters (2¾) inches.

These buckled and corrugated plates are mainly used to support pavements of highway bridges, and the trough plates to carry railway ties in ballast.

H-sections have been rolled in America for a few years only, but they have been procurable from Europe for some time. They make excellent small columns for highway bridges and can be used for diaphragms of heavy built columns. They are procurable up to fourteen (14) inches in depth by fourteen (14) inches in width. It is probable that they will be largely used in future by bridge designers.

Tees are very seldom required in bridge designing. Formerly they were employed for plate-girder stiffeners, but of late years they have been superseded almost entirely by angles.

Reinforcing bars are made of various types and sections, some good and others but little better than plain bars, of which many as yet are used for reinforcing—especially in Europe. The best kinds are the corrugated ones, but there is a choice between these, those having transverse corrugations being preferable to those having longitudinal corrugations. Square, twisted bars are inferior to corrugated ones, and it is doubtful whether they are superior to plain bars. Reinforcing bars are rolled in both high and medium steels, the principal advantage of the former being an economy in weight of metal, and the advantages of the latter a greater facility for fabricating in the field, a less brittleness, and occasionally a trifling saving in pound price. The author's invariable practice is to use medium steel for reinforcing bars.

Besides the preceding sections there are employed sometimes in bridgework the following: sheet-piling sections, bulb angles, round-back angles, rail-guard angles, square-root angles, and hand-rail tees, all of which can be found thoroughly illustrated and described in the handbooks of the various manufacturers of rolled metal.

CAST STEEL

The composition of cast steel varies but little from that of rolled medium steel, except that the carbon content and the permissible percentages of certain impurities are a trifle higher. In strength it lies about half-way between the medium and the high steels. For many years it was almost impossible to procure cast steel suitable for bridgework, owing mainly to blow-holes and other serious flaws in the castings; but today it is otherwise, for now one can count upon obtaining satisfactory castings without an undue amount of trouble. Unfortunately, though, the high price of steel castings, which until lately appeared to be unavoidable, has prevented their adoption to any great extent in bridge building; but there is a tendency today on the part of the manufacturers to substitute cast steel shoes and pedestals for built ones, and the pound prices for machinery castings have been lowered. It is now feasible to rivet steel castings to other metal work without running any serious risk of cracking them. They are highly desirable for certain parts of movable bridges, and their wider adoption is probably only a matter of time. It is by means of steel castings that the articulation of compression chords of bridges may be accomplished—which, in the author's opinion, is a *desideratum* of great importance for the science of bridge designing. A reference to this point will be found in Chapter XXII.

Cast Iron

As a rule, it is best to bar out cast iron from bridge building; nevertheless there are places where it may legitimately be employed—for instance, in heavy base-plates that rest on masonry and which are used for the purpose of distributing great loads over large areas, in counter-weights, in wood-washers, and in certain bearing blocks for operating machinery. None but the very best quality of cast iron should be employed for bridge building, excepting of course in counter-weights where weight is the sole *desideratum.*

Wrought Iron

Wrought iron nowadays is rarely used in bridge building. Its sole function there is for the manufacture of loop eye-bars, which have to be welded. Wrought iron is greatly superior to steel for welding purposes; hence it is still employed occasionally for hangers and suspenders in highway bridge building. It has another feature of superiority to steel in that it resists corrosion far better. If it could be purchased at about the same price as steel, it would be advisable to adopt it in places where resistance to deterioration rather than strength is the main function of the metal; for instance, in the shells and bracing of cylinder piers, and where the metal is to lie in or close to salt water. The wrought iron employed in bridge building some twenty-five or thirty years ago was generally a most superior metal, as is evidenced today by its demand for blacksmith shops when the old structures are removed.

Wire Rope

There is an unfounded prejudice on the part of many engineers and users of bridges against the adoption of wire rope in bridge building; for when it is of the proper quality, it is just as desirable and useful as any other of the materials employed therein. For movable spans it is eminently suitable, as it affords a cheaper and more reliable means of operation than does the rack-and-pinion method. None but the strongest and most pliable wire ropes manufactured should be used in the building of bridges; because the difference in price between a good and an inferior rope is a bagatelle.

Wire ropes are made with both hemp and wire centres. The former are generally the better for bridgework, unless the feature of non-stretching is of importance. All wire ropes will stretch more or less under use, but those with hemp centres far more than those with wire centres. On the other hand, the latter require much larger sheaves than the former, and these details are expensive. For the cables of suspension bridges the bending of the rope is not an important consideration, hence wire centres are advisable, not only because of their greater strength, but principally because the stretching of the cables would be objectionable. Such stretching is generally small in standing cables; nevertheless it does exist, for the component strands tend to close up even in stationary ropes. Every engineer who purposes using wire rope in his constructions should not only post himself thoroughly about the qualities and characteristics of the different kinds procurable, but also should familiarize himself with the correct methods of figuring and combining the various stresses to which they are subjected by both direct load and bending.

WIRE

Wire is used for bridgework also in reinforced concrete and in mattresses. For the former, strength is the prime requisite, but for the latter it is pliability. Wire mesh is employed in concrete piles and for cheap fences or railings of highway bridges.

COPPER

Copper is utilized but little in bridgework. Its use is confined mainly to electric wiring of operating machinery and to building the gutters and down-spouts of machinery houses.

BRONZE

Bronze is used in bridgework only for high-pressure bearings; for instance, in the pivot sockets of centre-bearing swing spans, or sheave-journals of vertical lift bridges.

BABBITT METAL

Babbitt metal is sometimes adopted for machinery bearings, but its principal use in bridge building is for filling sockets in the attachment thereto of wire rope. It is melted and poured in between the spread and turned-back ends of the individual wires, thus preventing their pulling through the eye. Of late zinc has largely replaced Babbitt metal for filling sockets.

TIN PLATE

Tin plate is employed in bridge building only in the covering of the roofs of machinery houses. It is important to specify the best quality that the market affords, as there is no economy in using an inferior article.

LEAD

Lead is sometimes adopted to procure an even bearing between metal and masonry, but as the pressure makes it flow into the interstices of the stone, an objectionable splitting tendency results, hence the practice is not to be recommended. Lead is also employed to exclude water from the expansion joints in concrete floor-slabs; but otherwise it is not much used in bridgework. As there is no other use for lead in bridge building, that metal may properly be excluded from the list of materials employed in bridgework.

PAINT

This is such an important material for bridges that an entire chapter (No. XXXIV) is devoted to its discussion.

PAINT-SKINS

Paint-skins are utilized for filling small spaces in metal work before the protective covering is applied. Asphaltum is sometimes used instead.

ASPHALT

Asphalt is employed in bridge building mainly for pavements; but when mixed with pitch, it is used between the layers and in the cracks of planking and in coating bolt-holes in wood.

PITCH

Pitch is used in bridges for protecting wood and for caulking caissons.

OAKUM

Oakum also is used for the latter purpose.

FELT

Felt is utilized in bridge building mainly for expansion joints in concrete and for placing, when covered with hot asphaltum or pitch, between the two thicknesses of plank flooring, or under pavements to prevent leakage.

ASBESTOS

Asbestos in the form of cloth is sometimes employed instead of felt for the expansion joints in concrete work.

PLASTER

Plaster is used in bridges solely for the walls of the machinery houses. Only the very best quality that the market affords should be purchased, because the vibration of the machinery has a tendency to loosen the plastering.

TIMBER

Timber was formerly used in bridge construction far more extensively than it is today, entire bridges—both substructure and superstructure—being built of it almost exclusively, but nowadays its employment is gradually being reduced. This is because of three good reasons: first, its perishability; second, its increasing scarceness, and, third, its consequently augmented price. In the days of the Howe truss bridge it was the builder's most important material, for the trusses of that type were constructed mainly of timber; but today wooden bridges are built only in the most remote districts and in communities where there is not sufficient money available for steel or concrete structures. It is still employed largely for trestles, both railway and highway, and will continue to be so used until the price of timber becomes prohibitory, the day for which is not far distant. It is employed largely for piling, but even there it is being gradually replaced by reinforced concrete.

The kinds of timber most used in bridge building are the long-leaf yellow pine of the Southern States and the Douglas fir of the Pacific Coast. Both are excellent. Oak used to be employed a good deal for the track ties of railroad bridges and for the flooring of highway structures; but it has gotten into disfavor among bridge builders for several reasons, viz., its tendency to warp and split, its liability to dry rot, the exhaustion of the better kinds, and the augmented price of the inferior species. The good oaks are the white, cow, chincapin, post, burr or overcup, and live oaks. The bad ones are the red, Spanish or water, black, black-jack, and pin or yellow-butt oaks.

Short-leaf yellow pine is allowable in bridgework only where it is to be kept permanently under water, as in cribs and caissons of piers. It is too short-lived and brashy to be used elsewhere, unless it be for the interior finishing of machinery houses, and for this purpose other materials are generally preferable.

Cypress of certain kinds is valuable for piling because of its durability, straightness, and great length; but for most places it is a rather expensive timber to adopt. Red, black, and yellow cypresses are good, but white cypress is not.

Pacific Coast cedar is an excellent timber for piling. It could be used for other purposes, and might be, were it not that Douglas fir is preferable and just as available.

Timber has, of late years, become so scarce and expensive that it generally pays to treat it, the best preserving process for bridge timber being that of creosoting. Specifications for this method of treatment are given in Chapter LXXIX.

Timber still continues to be used largely for highway bridge floors, but it is gradually being replaced by steel or reinforced concrete for the joists and by reinforced concrete for the planks. It will long continue to be employed for the decks of such structures, as creosoted yellow-pine blocks make the best kind of pavement for bridges. Timber is still the principal material for building the cribs and caissons of piers, but steel shells filled with concrete have been employed occasionally for more than a quarter of a century; and today reinforced concrete caissons have been established as more than a possibility. Timber for ties in railroad bridges will probably hold its own for many years, owing to the cushioning effect of that material, but the time will certainly come when something else must be used.

Brush

Brush is employed in bridgework for the building of mattresses to protect the piers and the river banks from scour. The best kind of wood for brush is willow, the requirements being strength, toughness, and pliability. Specifications for it are given in Chapter LXXIX.

Stone

There are two general classes of stone used in bridge building, viz., masonry stone and broken stone for concrete. It was only a few years ago that piers and arches were built almost exclusively of stone masonry, but today nearly all of them are being constructed of concrete. The best kind of stone for masonry is granite, but at the same time it is nearly always the most expensive, owing to the high cost of dressing. The better kinds of limestone are the next best stones for masonry, and are the ones generally employed. Sandstones usually are the poorest, but there are certain metamorphic sandstones that are as strong and as durable as granite, but at the same time they are about as expensive to work, owing to the absence of cleavage planes. Specifications for masonry stone are given in Chapter LXXIX.

The main requisites for concrete stone are that it shall be hard, clean, and durable. A certain amount of impurity does not injure it materially; but, in general, it may be stated that the cleaner the stone the better will be the concrete, notwithstanding the fact that certain experiments have shown that concrete made of dirty stone is stronger than that made of the same kind of stone after being washed. The impurity generally consists of clay. If this is mixed uniformly throughout the mass, it will do but little harm, and may even apparently do some good; but, unfortunately, it generally adheres in small lumps to the stone, and these certainly reduce the resistance to shear and injure the tensile strength. Stone dust, or, as it is sometimes termed, "quarry dust," is not an impurity, for it acts like sand in the concrete. Smooth stone is inferior to rough stone for making concrete. Stone is occasionally used for the paving of bridges; but, owing to its great weight, it generally involves too much expense for steel superstructures. It is eminently suitable for masonry arches or reinforced concrete bridges, where a large dead load is not objectionable.

BRICK

Brick is not much employed for bridge construction in America; nevertheless well-built piers of hard-burned brick laid up in rich Portland cement mortar are truly first-class constructions. They can be adopted advantageously in places where stone and gravel are either not procurable or very expensive. Moreover, brick-bats, when broken small enough, make pretty fair concrete, provided that no soft bricks be allowed.

GRAVEL

Gravel is suitable for the principal ingredient of concrete, although it does not develop as great strength as broken stone. This may be due to the smoothness of its surfaces or to the lack of locking power, which broken stone possesses to an eminent degree. A certain proportion of gravel may be added to broken stone with advantage, in that the addition will decrease the percentage of voids in the mass and thus lessen the quantity of cement required. Gravel should be clean and free from all impurities of a character that would be injurious to the concrete, such as chips of wood and pieces of bark. The rougher the pieces of stone and the more varying their sizes the better. In general, it is preferable to use broken stone for concrete, when it is procurable at reasonable expense, and gravel in other cases; but when a bed of sand and gravel mixed in about the right proportions for concrete is available, the temptation to employ the mixture is hard to resist.

Gravel that is too dirty for concrete can often be washed at reasonable expense and thus rendered suitable.

SHELLS

Oyster shells are sometimes used instead of broken stone or gravel for making concrete; but the practice is reprehensible, for such shells are not strong enough. The strength of a concrete is a direct function of the strength of its main ingredient, hence an engineer should avoid trouble by refusing to employ a weak material therefor, even if all better materials available be much more expensive. Of course, shell concrete can be employed as a last resort; but it should be loaded lightly, and, in many cases, more of it should be used than would be the case were a first-class material adopted.

CINDERS

Cinders are sometimes used in the concrete floor-slabs of bridges, but they are just as objectionable as shells, if not more so. The only excuse for employing cinder-concrete is to reduce the dead load; and this reduction is obtained only by leaving voids in the mass—which, for obvious reasons, is objectionable.

SAND

Sand is a very important constituent of concrete or mortar. The use of a poor sand will often reduce the strength of the product as much as fifty per cent. Sand should be coarse, sharp, and clean. Very fine sand or quicksand in concrete is liable to ruin it. Sand can often be improved materially by washing, as that process, when properly applied, carries off most of the impurities. Sand with smooth, rounded grains does not make good concrete or good mortar. The best sand for these that is found in nature is one that has sharp corners and rough surfaces, with grains of all sizes from coarse to very fine, and in which the percentage of voids is a minimum.

CEMENT

The importance of using first-class cement for both concrete and masonry cannot be too forcibly emphasized. A barrel or two of bad cement might be the means of causing the destruction of a bridge and untold disaster in consequence. There are in the market today plenty of really good cements; hence there is no excuse for ever employing a poor one. The foreign cements used in America, as a rule, are more reliable than American cements. There are two reasons for this: first, the manufacturers thereof have had more experience, and, second, the long sea voyage allows ample time for thorough hydration. A great deal of American cement is shipped hot from the mill to the consumer; and if he has it properly tested, he will often find that it will not stand the steaming and boiling tests, and that the pats made of it will either crack or fail to harden as they should. The author has had much trouble and unpleasantness with both substructure contractors and the manufacturers of cement on this account; and he expects to have more in the future because of the inexcusable custom of putting on the market cement that has not weathered. How seriously such cement injures the concrete is hard to say; but no conservative engineer will be guilty of taking any chance by using an inferior article when one of known excellence is procurable.

In the old days of bridge building much natural or so-called Rosendale or Roman cement was employed; but it was always inferior in strength to Portland cement, and as the price of the latter has been reduced to a very reasonable figure, there is no further need of ever considering the natural cements for bridge construction.

The specifications for cement given in Chapter LXXIX are quite reasonable in their demands, and at the same time they will provide a sufficiently good material for all purposes. Any cement that develops its strength very rapidly should be regarded with suspicion, because, if tested for a period of several months, it will almost certainly show a decided drop; and this loss may or may not be recovered later. Some cements set too rapidly for bridgework, hence pains should be taken to determine quite often the times required for setting. New brands of cement need more thorough testing than old brands of established reputation. No American cement should ever be used without testing, and European cements should be tested also, if practicable; but if one is in a hurry, he would run no great risk by using without testing for more than one day any European brand of cement of established reputation, provided each barrel be carefully examined for injury by water. The author once employed on one piece of work some sixty thousand barrels of a well-known brand of German Portland cement without having to reject a single barrel, excepting a few in which the contents were caked from exposure to rain.

LIME

No man who deems himself a bridge engineer will ever consider for a moment the use of lime in any of his structures. It is a material that is entirely unsuitable for his purpose. The only reason that could possibly be advanced for its employment is economy—and such economy would certainly be false. When mixed with cement, lime lowers the strength of mortar very rapidly, and when employed alone it is totally unfit for use in any kind of engineering construction.

CHAPTER IV

ALLOY STEELS IN BRIDGEWORK

The use of alloy steels for bridgework is such an important matter that it seems advisable to devote to it an entire chapter, thus segregating them from the ordinary materials of bridge construction, which were treated in the preceding chapter.

Alloy steels are almost always more expensive than ordinary steel, but they are generally stronger; hence, in order to effect any certain purpose, it usually does not require as much weight of metal as if the said ordinary steel were employed. The proper adjustment of the lesser weight to the greater pound price will determine the economy or lack of economy in employing the alloy, excepting only in those minor cases in which some other characteristic than mere strength, such as hardness, resistance to abrasion, or smallness of volume, necessitates its adoption.

Late in 1914, in compliance with an invitation from the International Engineering Congress of the San Francisco Exposition, the author prepared a paper entitled "Alloy Steels in Bridgework"; and as that paper aims to cover concisely in a general manner the whole ground of the subject, it is herewith reproduced as follows almost in its entirety, omitting only the opening paragraphs. For the convenience of the reader, the system of numbering the figures and the tables has been changed to agree with that in the other chapters:

"It was but a dozen years ago that the alloy, nickel steel, began to be talked of seriously for bridge building. Before the days of medium steel, however, a few large bridges were constructed of special steels, notably in America, the Eads Bridge of St. Louis, Mo., and the Chicago and Alton Railway bridge at Glasgow, Mo. The latter was of Hay steel; and although the author worked for a short time in a subordinate capacity on the structure, he has forgotten the composition and the characteristics of the metal, except, perhaps, that it was rather high in carbon. Be this as it may, the matter is of no special importance; because that make of steel, as far as the author knows, was never again used in any important bridge.

"The term 'manganese steel' for bridgework is somewhat in the nature of a misnomer, for all bridge steel has to contain a certain amount of manganese (generally from 0.5 to 0.8 per cent) in order to make it workable in the mills and otherwise satisfactory; but when a bridge engineer uses the term, he means a steel very high in manganese, and, consequently, exceptionally hard. Such a steel or alloy is employed for rail-locks and for those parts of the operating machinery of movable spans where great resistance to both abrasion and shock is the principal *desideratum*.

"Chrome steel, an alloy of chromium with steel, the author has heard of as being used for this purpose, but not at all generally on account of its high price.

"Chrome-nickel steel has also been employed somewhat for special castings in bridge machinery, but its principal use is for the manufacture of aeroplanes, automobiles, transmission lines, and gearing.

"In most cases it is necessary to submit these various alloys to heat treatment in order to increase materially their hardness, elastic limit, and ultimate strength. The price for castings or forgings of such alloys generally varies from 8 to 13 cents per pound, according to the amount of shop-tooling required. Such prices, of course, are prohibitory for bridgework, excepting only for small but important parts of operating machinery and for details requiring great resistance to abrasion.

"Some manufacturers claim to be able to produce alloy steels having elastic limits as high as 250,000 lbs. per square inch; but it is impracticable to shop-tool them when the elastic limit exceeds 150,000 lbs., or when the ultimate strength is greater than 200,000 lbs. Such metal might possibly be required for bridge pins and their bearings in order to meet certain extreme conditions; but the probability of such requirement is exceedingly remote. Moreover, bridge engineers, as a rule, are loath to concentrate great stresses on members of very small cross-section because of the proportionately great effect thereon of any undiscovered small cracks or flaws which may exist in the metal.

"While nickel steel was talked of for bridgework in both Europe and America prior to 1902, it was not employed therefor. In that year and in 1903 the well-known consulting engineer and bridge specialist, Mr. Gustav Lindenthal, started some experiments upon the use of nickel steel for the eye-bars of the Blackwell's Island Bridge at New York City, reporting favorably thereon. Later, after trying hard to avoid its employment, the city authorities decided to adopt the alloy for the said eye-bars; and the bridge was constructed accordingly. This was the first actual use of nickel steel in bridgework.

"In 1903, before the city authorities just mentioned came to their decision, the author inaugurated an exhaustive series of experiments and investigations upon the subject of the suitability of nickel steel for bridge building in general and its economics therein. In spite of many trials and tribulations, and in the face of strong opposition and great discouragement, he succeeded, after more than three years, in completing most of the work which, at the beginning of his undertaking, he had laid out to do; and it required some three months more to digest the results and to prepare a report for his principals, who were the International Nickel Company. That corporation financed the entire undertaking, spending altogether upon it nearly $50,000.00. As soon as the report was completed, the author devoted several months to the preparation of a monograph upon the subject, and presented it to the American Society of Civil Engineers. After taking considerable time for deliberation, the Publication Committee of that Society rejected the paper without giving any reason therefor. Fellow members of the Society especially interested in bridgework urged a reconsideration of the matter—which caused the Committee to reverse its original decision, provided that the author would agree to cut down materially the volume of the original memoir. The reduction was accomplished by omitting some of the records of tests and all the diagrams and text relating to the economic study of bridges built wholly of nickel steel, retaining only those concerning structures built of mixed nickel steel and carbon steel. The original (rejected) memoir containing all records and diagrams is on file in the Society's library, where it can be consulted by any one interested in this subject. The paper in due time appeared in the Society's 'Proceedings' and was discussed by thirty or more engineers, both American and European; and finally it was published with the discussions in the 1909 'Transactions' of the American Society of Civil Engineers. Later the author of the memoir was awarded the Norman medal, because of its being the best paper presented in that year.

"The entire investigation proved (at least to the author's satisfaction as well as to that of a large majority of the engineers who entered into the discussion) that nickel steel is in every way a suitable metal for the manufacture of bridge superstructures, being just as reliable as carbon steel and from 50 to 70 per cent stronger. The correctness of this statement is proved by the fact that the alloy was used later not only in the Blackwell's Island Bridge before referred to, but also in the Manhattan Suspension Bridge at New York and in the Free Bridge at St. Louis. The last-mentioned structure was designed and engineered by Henry W. Hodge, Esq., one of America's most noted bridge specialists. Again, the new Quebec Bridge, which will contain the longest span in the world, viz., 1,800 feet, is being partially constructed of nickel steel.

"The author found by his experiments and investigations that it is perfectly feasible to produce commercially an eminently satisfactory nickel steel for bridgework, having a minimum elastic limit of 60,000 lbs. per square inch, a minimum ultimate tensile strength exceeding 100,000 lbs. per square inch, and an elongation in eight inches of fifteen (15) per cent. The actual extra cost per pound for this metal, delivered at bridge site, as compared with ordinary carbon bridge steel, he figured should not be more than 1.5 cents. Unfortunately, however, the steel makers and bridge manufacturers, being opposed, naturally, for pecuniary reasons, to fundamental innovations in their business, have not responded to the call of the bridge engineers for a nickel steel of great strength at a moderate price, preferring to continue without interruption the production and manufacture into bridges of the cheaper carbon steel to which they are accustomed. For years they have made a practice of refusing to guarantee for nickel steel an elastic limit of more than 50,000 lbs. per square inch; and they have asked therefor an excess pound price of from 1.5 to 2.0 cents. Mr. Hodge paid 1.65 cents per pound extra for his nickel steel in the St. Louis Bridge; and the price named for the alloy in the new Quebec Bridge was so high that it was found economical to use it only for the truss members of the suspended span and the cantilever arms. It is true that by adopting carbon steel instead of nickel steel for the anchor arms of a cantilever bridge, the weight of those arms is increased, and, in consequence, the stresses on the anchorage metal and the uplifts on the anchor piers are reduced, but these results could probably be obtained more economically in some other manner—for instance, by adopting a ballasted floor for the tracks on the anchor arms only.

"In the case of his proposed bridge across the entrance channel to the harbor of Havana, Cuba,* the author, by great effort, succeeded in persuading the Carnegie Steel Company and the American Bridge Company to agree to furnish him with nickel steel having an elastic limit of 55,000 lbs.; but the extra pound price demanded for the manufactured metal was 2.5 cents. With these figures it was an exact stand-off between nickel steel and carbon steel for both the suspended span and the main structure as a whole; but as the said suspended span will have to be built on barges, floated to site, and raised by wire ropes to final position, the author concluded to adopt the alloy for that portion of the superstructure. He found also that it would involve a trifling economy to use it in the cantilever arms, but not in the anchor arms; hence he has decided to follow the same course as the designers of the new Quebec Bridge did in relation to their great structure.

"The compositions of the various classes of nickel steel for bridgework recommended by the author, in view of the results of his experiments, were as given in the following table:

TABLE 4a

COMPOSITIONS OF THE VARIOUS CLASSES OF NICKEL STEEL FOR BRIDGES

Ingredients	Percentages		
	Rivet Steel	Plate-and-Shape Steel	Eye-bar Steel
Nickel..........	3.50 (3.25 to 3.75)	3.50 (3.25 to 3.75)	4.25 (4.0 to 4.5)
Carbon..........	0.15 (0.12 to 0.18)	0.38 (0.34 to 0.42)	0.45 (0.4 to 0.5)
Phosphorus......	0.03 Max.	0.03 Max.	0.03 Max.
Sulphur.........	0.04 Max.	0.04 Max.	0.04 Max.
Silicon..........	0.04 Max.	0.04 Max.	0.04 Max.
Manganese.......	0.60 (0.55 to 0.65)	0.70 (0.65 to 0.75)	0.80 (0.75 to 0.85)

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* See Fig. 52a for photograph of proposed structure.

"The rivet steel specified is as high in carbon as it is practicable to go, in view of the fact that rivets must not be too hard to cut out when badly driven.

"The carbon percentage in the plate-and-shape steel is as high as will permit of the metal being worked satisfactorily in the shops.

"The percentage of nickel in both the rivet steel and the plate-and-shape steel is as high as considerations of both economy and workability allow.

"As the phosphorus, sulphur, and silicon are in the nature of impurities, their percentages are kept as low as is consistent with economy in smelting; because it is expensive to reduce the said impurities below the figures shown—in fact, even these have raised objections among some steel makers.

"In eye-bar steel it is permissible to make the metal harder than in plate-and-shape steel, because the shop tooling on eye-bars is small in amount and of simple character; hence the percentages of both nickel and carbon, adopted above therefor, are quite high. The extreme limit specified for the nickel, viz., 4.5 per cent, causes the alloy to approach the brittle zone, which begins at some yet undetermined figure between 4.25 and 5 per cent and ends at 20 per cent. This brittle zone was discovered by three English metallurgists, Messrs. Carpenter, Hadfield, and Longmuir, and was described by them in November, 1905, in a paper read before the Institute of Mechanical Engineers of England. It is claimed, however, by some American experimenters that the use of more than five (5) per cent of nickel does not of necessity make the steel brittle; hence it is likely that the percentage of carbon has some influence on the brittle zone of alloy steels containing more than the said five (5) per cent of nickel. Be this as it may, though, an engineer should test carefully for brittleness his eye-bar steel, if he employs in its manufacture nickel in any greater percentage than 4.25.

"As shown in the preceding table, the amount of manganese in nickel steel is graded to meet the requirements of both strength and hardness. It varies from 0.6 to 0.8 per cent.

"In the memoir, 'Nickel Steel for Bridges,' are given twelve diagrams of weights of metal per lineal foot of span, covering all lengths from 20 feet for plate girders up to 1,800 feet for cantilever main openings. In Figs. 4*a* and 4*b* of this memoir are reproduced two of the most interesting and useful of those diagrams covering double-track, through, pin-connected, Petit truss spans and double-track, through, pin-connected, cantilever bridges of Type A (see Fig. 55*aaa*). The weights given are for structures built both wholly and partially of nickel steel, and for those composed entirely of carbon steel.

"Following the twelve weight records in that memoir come some fifty economic diagrams, which show for all span lengths up to the before-mentioned limit for both riveted and pin-connected structures, for all

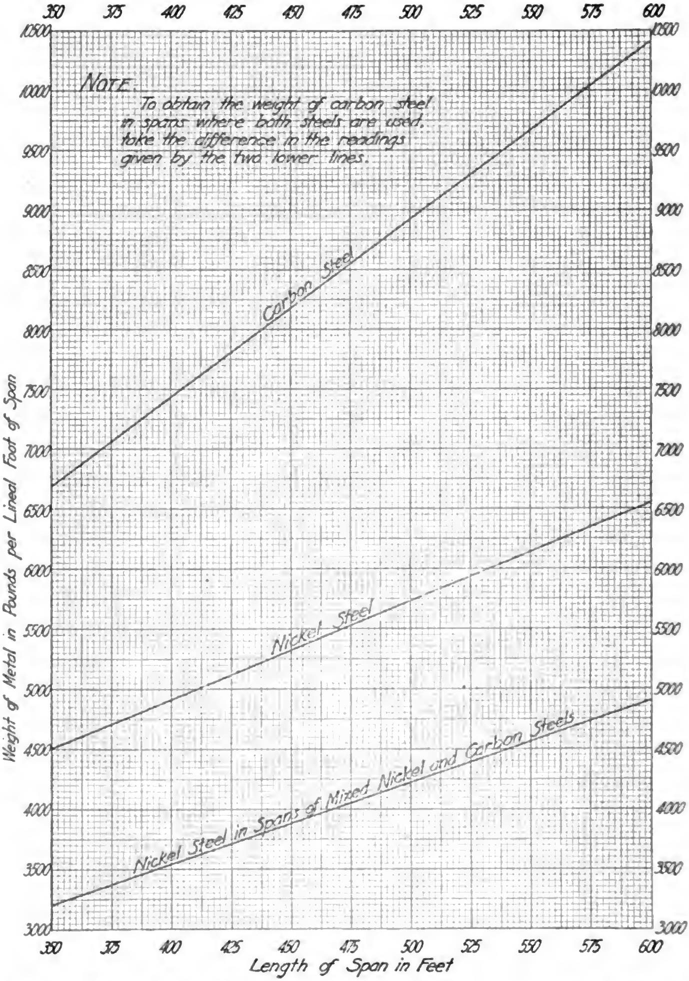


FIG. 4*a*. Weights of Double Track, Through, Pin-connected, Petit-truss Spans of Carbon Steel and Nickel Steel.

possible conditions of the metal market, and for all probable variations in pound prices between the manufactured nickel and carbon steels, the comparative costs of bridge superstructures built of carbon steel and of mixed nickel steel and carbon steel. With these fifty diagrams at hand and all the necessary conditions given, it is only a minute's work to determine for any case whether it would be more economic or not to adopt nickel steel—also what the saving, if any, in expense would be.

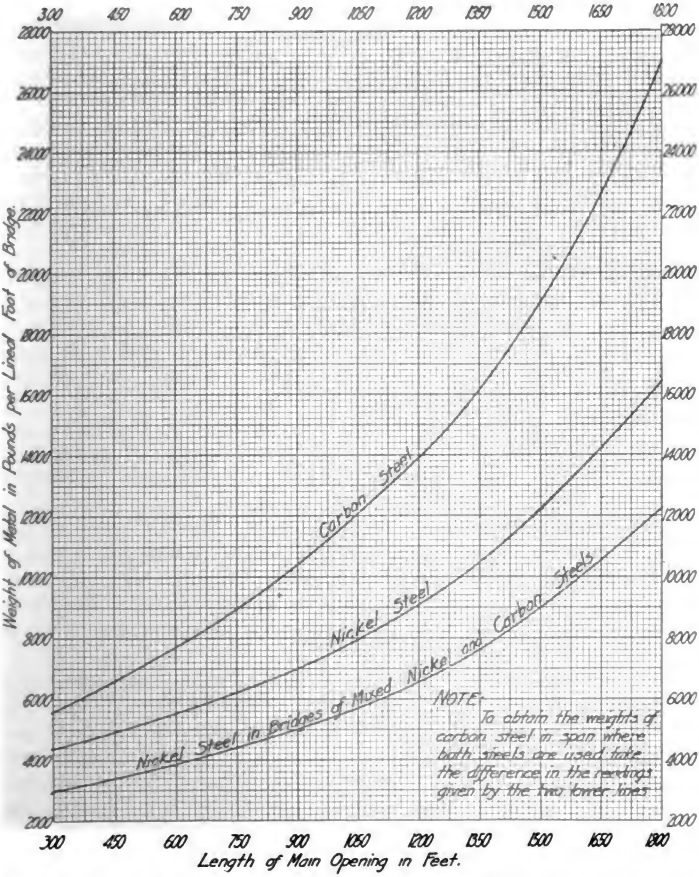


FIG. 4*b*. Weights of Double-track, Through, Pin-connected, Cantilever Bridges of Carbon Steel and Nickel Steel.

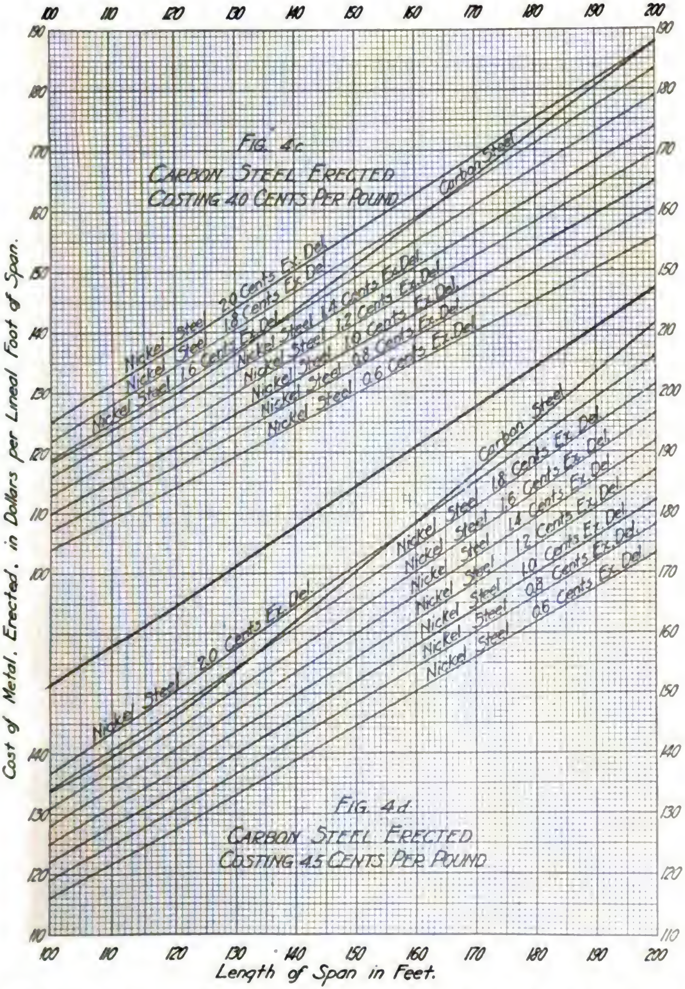
"Figs. 4*c*, 4*d*, 4*e*, and 4*f*, chosen at random, are specimens of the said fifty economic diagrams, the first pair being for double-track, through riveted, Pratt truss spans, with the price of carbon steel erected at 4 and 4.5 cents per pound, respectively; and the second pair for double-track, through, pin-connected, cantilever bridges of the most usual type, with the price of carbon steel erected at 4.5 and 5 cents per pound, respectively. These prices for carbon steel erected are about the average ones that govern today in various portions of the United States.

"Fig. 4*g* is an important and interesting diagram. It shows the probable weights of metal per lineal foot of superstructure for very-long-span, double-track-railway, cantilever bridges built of carbon steel and of nickel steel (or of mixed nickel and carbon steels). It indicates also the extreme practicable limit of length of main opening for such bridges for each kind of steel. This limit is a matter of judgment, being determined by the greatest weight of metal per lineal foot of span which it would be advisable to use for the structure under consideration. From the diagram it will be seen that if 1,800 feet be assumed as the present practicable limit of span-length for carbon steel bridges, the corresponding limit for nickel steel bridges will be about 2,300 feet; or, if it be assumed at 2,000 feet, the corresponding limit for nickel-steel construction will be 2,600 feet. It is safe, therefore, to conclude that the adoption of nickel steel for bridges would lengthen the practicable span length for cantilevers fully 500 feet.

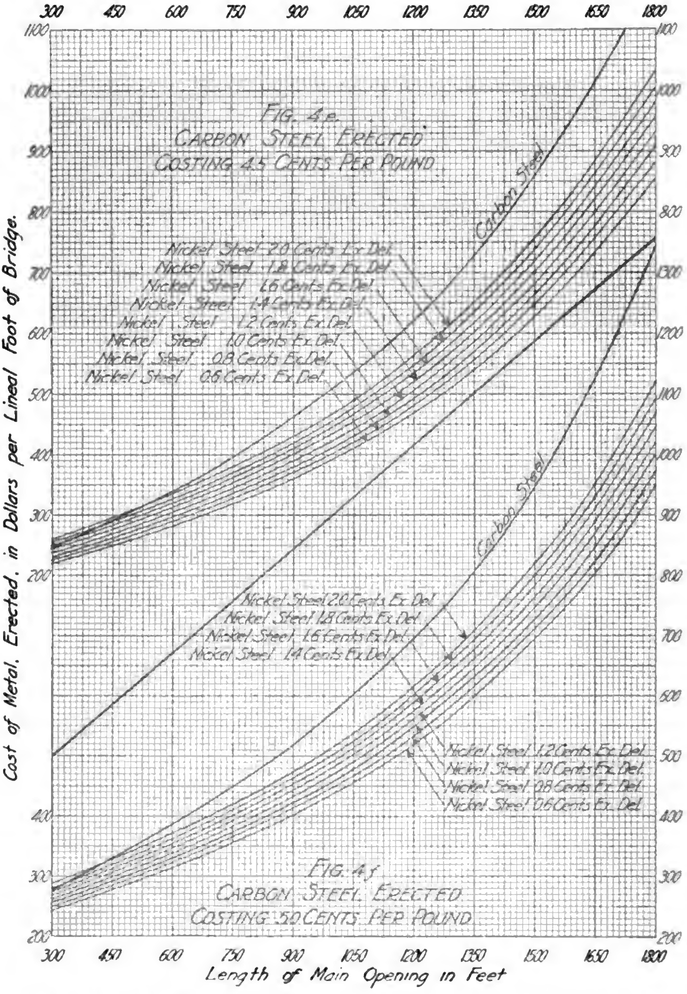
"In concluding his paper on 'Nickel Steel for Bridges' the author wrote as follows:

"'Summarizing the results of this entire investigation, it is evident that nickel steel is in every way fitted for bridge construction, in that it is strong, tough, workable, and reliable; moreover, its adoption would effect a decided economy. This economy would increase in the future as the cost of nickel decreases and as the shops become more accustomed to the fabrication of the new alloy.'

"The preceding was written in 1907, and the predications made have been only partially realized; for while, as before indicated, several large bridges have been built of nickel steel, the manufacturers have not been willing to quote reasonable prices for the alloy. If there were any one available upon whom to unload rejections, as there is in the case of carbon bridge steel, the steel makers would quite willingly quote more reasonable figures for nickel steel; but the constant dread of being left with a large lot of unsalable alloy steel on their hands militates against their so doing. It is only by having engineers create a large demand for the alloy, thus initiating competition in its production, that a reasonable pound price for it can be established. In confirmation of this statement is the fact that when Mr. Hodge called for bids on nickel steel for his great St. Louis Free Bridge, he received and accepted a tender of an excess pound price of 1.65 cents, which is not far from the 1.5-cent limit set by the author in his memoir.



Figs. 4*c* and 4*d*. Comparative Costs of Double-track, Through, Riveted, Pratt-truss Spans of Carbon Steel and Mixed Nickel and Carbon Steels.



FIGS. 4e and 4f. Comparative Costs of Double-track, Through, Pin-connected, Cantilever Bridges of Carbon Steel and Mixed Nickel and Carbon Steels.

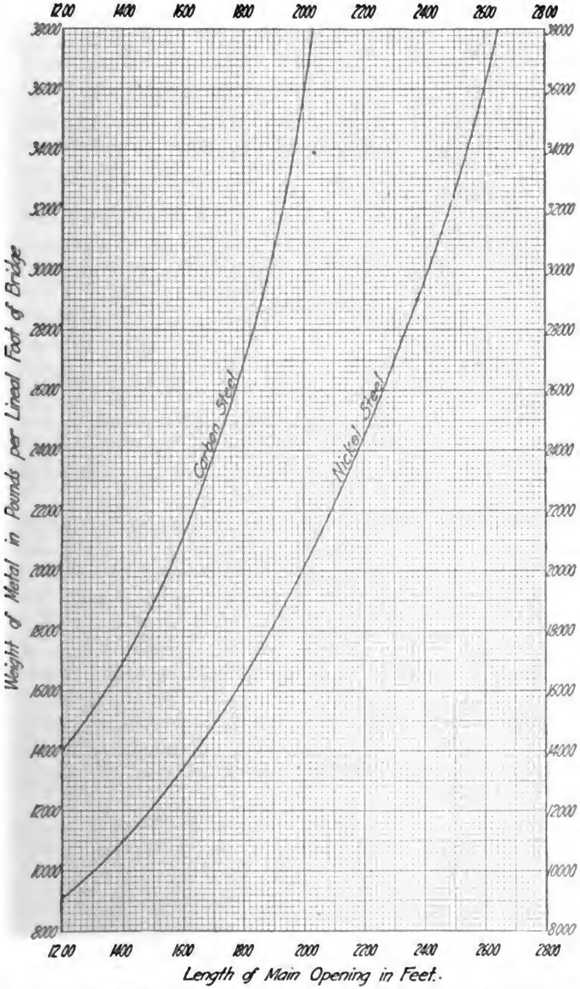


FIG. 4g. Probable Weights of Very-long-span Cantilever Bridges of Carbon Steel and of Nickel Steel.

"It is claimed by several recognized authorities that it is practicable to produce good and perfectly satisfactory nickel steel by putting ferro-nickel into the charge instead of the metallic nickel, thus avoiding all the expense of refining. This would reduce by two-thirds the cost of the nickel content in the alloy; and as that content is the main cause of the high price of nickel steel, it is evident that the employment of ferro-nickel in the smelting would make the cost of the product so reasonable that in a few years it would supplant carbon steel entirely, even for bridges of the shortest spans. Nickel producers used to claim that it is absolutely necessary to employ pure nickel in the smelting, for the reason that ferro-nickel usually contains quite a percentage of copper—a substance totally destructive to steel; but, on the other hand, those who are in shape to put ferro-nickel on the market (and some others also) maintain that the copper and all the other injurious substances contained in the ferro-nickel can readily and cheaply be worked out during the processes of smelting and rolling. Moreover, copper is no longer the bugbear to steel makers that it was a few years ago; for it is now practicable to manufacture good, workable steel containing three (3) per cent of that element. Decidedly, it is of the utmost importance to both the engineering profession and the business world to determine without delay and beyond the peradventure of a doubt whether it is feasible to use, on a commercial scale, ferro-nickel in the manufacture of nickel steel.

"There is a fact concerning nickel steel known to the profession, but which, as far as the author can learn, had not until a very short time ago been stated in print, viz., that the Pennsylvania Steel Company has obtained control of an iron deposit containing a small percentage of nickel, and is, consequently, able to place upon the market a low-grade nickel steel at a reasonable excess cost above that of carbon steel. This steel has been denominated by its makers 'Mayarí Steel.' It is a natural alloy of nickel-chromium steel, containing from 1% to 1.5% of nickel and from 0.2% to 0.75% of chromium, with sulphur below .04%, phosphorus below .03%, and manganese as desired. The carbon range is from .03% to 1.5%, depending upon the application of the steel.

"The ore comes from a deposit of some 25,000 acres at Mayarí in the Province of Oriente on the Island of Cuba. It is estimated that there are 500,000,000 tons of this ore in sight.

"Mayarí steel is made only by the Pennsylvania Steel Company and the Maryland Steel Company. By a slight modification of the open-hearth process it is produced without the necessity of adding alloying elements in the furnace or ladle. Like other nickel steels it offers greater resistance to corrosion than do the ordinary carbon steels.

"Desiring to obtain for the preparation of this memoir some authentic information concerning the new alloy, the author wrote to the Pennsylvania Steel Company asking certain questions about it, and received in reply a letter from J. V. W. Reynders, Esq., C. E., the Vice-President of the Company, from which the following is an extract:

"'Our principal experience on Mayari steel in bridgework has been in connection with the manufacture and fabrication of the large bridge which is to span the Mississippi River at Memphis. So much steel has now been made for this contract that we have accurate information on the properties which we can develop.

"'On this bridge alternate quotations on carbon and alloy steel designs were submitted, the specifications for the alloy steel, outside of rivet and eye-bar steel, being as follows:

Tensile strength................	85,000— 100,000 lbs. per sq. in.
Elastic limit, not less than........	50,000 lbs. per sq. in.
Elongation in 8 inches not less than	$\frac{1,600,000}{\text{T.S.}}$
Reduction of area, not less than...	30.0%

"'The manganese in the steel was limited to .80%, silicon to .15%, carbon to .40%, and a minimum of 1.20% nickel was required, but the individual bidder was allowed to select his own analysis except as it might be limited by these general figures. No limits were given for chromium or vanadium.

"'We quoted on the basis of using a steel made from the Mayarí ore which we import from our mines on the north coast of Cuba, and on this basis the contract was awarded us.

"'As you doubtless know, this Mayarí ore lies just under a thin top soil in a comparatively thin bed of great area. The ore contains naturally a large amount of moisture, a part of which is in the combined form. It has been our practice before shipping this ore to the United States to run it through rotary nodulizing kilns, which agglomerate the fine ore and drive out the moisture. The nodulized product carries about 57% of iron.

"'By a selection of the ore, steel can be produced with a uniform nickel content, which may be varied at will between quite wide limits. It has been found, however, that a content of approximately 1.40% is sufficiently high for bridge steel for most purposes. The steel is normally produced with only the usual additions in the open-hearth furnace, although occasionally a small amount of chromium is added.

"'The following are typical tests of large-size angles, varying from 8" × 6" × 1" up to 8" × 8" × 1⅛":

TABLE 4*b*

Tests of Large Size Angles of Mayarí Steel

T. S.	E. L.	Elong. in 8 Inches	Reduc. of Area	C.	Mn.	Ni.	Cr.
lbs.	lbs.	%	%	%	%	%	%
95,580	64,700	16.8	45.5	.36	.62	1.27	.36
91,400	60,130	19.5	44.4	.32	.68	1.45	.44
95,440	62,300	18.0	46.7	.34	.75	1.51	.38
94,400	54,300	17.0	50.4	.35	.71	1.48	.43
93,240	55,300	21.5	51.3	.34	.75	1.37	.43
96,740	60,540	16.3	43.3	.32	.75	1.49	.40
98,420	61,060	17.0	48.9	.37	.78	1.31	.42
91,700	56,960	19.0	56.3	.31	.68	1.45	.44
94,180	58,130	20.5	51.5	.30	.77	1.48	.47

"'The following tests are on plates, both universal and sheared. In many cases you will note that the thickness is extreme.

TABLE 4c

TESTS OF PLATES OF MAYARÍ STEEL

Thickness	T. S.	E. L.	Elong. in 8 Inches	Reduc. of Area	C.	Mn.	Ni.	Cr.
ins.	lbs.	lbs.	%	%	%	%	%	%
1½	99,600	64,660	17.5	41.9	.29	.77	1.36	.31
1¼	94,300	62,270	18.0	33.6	.28	.75	1.41	.37
1½	98,820	63,080	17.0	34.7	.28	.72	1.49	.49
⅞	90,040	55,270	21.0	45.5	.28	.67	1.57	.31
1⅛	91,550	62,210	18.5	46.8	.30	.66	1.41	.40
¾	94,060	56,030	18.5	47.1	.27	.61	1.42	.33
1⅜	90,300	57,540	19.0	53.6	.29	.71	1.53	.39

"'The phosphorus in all these heats will average less than .02% and the sulphur averages about .03%, the specified limit for each being .04% with an allowance of 25% thereof for check analysis of the finished material.

"'In the specifications changes in the elongation and reduction of area are allowed for steel running over 1 inch thick.

"'Below are given the specifications for full size eye-bars and the results of a test on a sample 14″ × 1-23/32″ bar:

	Required	Obtained
Tensile strength	80,000 lbs. min.	88,200 lbs.
Elastic limit	47,000 lbs. min.	51,700 lbs.
Elong. in 20′	10%	12.7%
Reduc. of area	..	42.0%

"'It has been our experience that this alloy steel works quite as well in the shops as any other steel with which we are familiar, making due allowance, of course, for the increased toughness. We find that it is easier to work than 3¼% nickel steel.

"'It is difficult to give an exact extra which we would charge for rolled sections or plates of Mayarí steel over the market price of similar sections of carbon steel. We do not expect however that it will be necessary at any time to charge an extra of more than one cent per pound. The excess price for manufactured bridges depends on so many circumstances that it is almost impossible to give any figure. It will vary greatly, of course, as the relative proportions of carbon and Mayarí steels in the finished bridge are varied.

"'With regard to quantity, we expect shortly to be in a position to produce from 18,000 to 20,000 tons per month of Mayarí steel shapes, if necessary; but, even for the present, it is safe to say that we can meet any reasonable demand.'

"Judging from Mr. Reynders' approximate quotation for the rolled metal and from previous experience with nickel steel, the author concludes that the finished metal work is likely to cost as much as one and a half cents per pound in excess of the market price of the corresponding carbon steel work. This is just what he first estimated would be the limiting excess pound price of nickel steel having an elastic limit of 60,000 lbs., when the excess price of the rolled material was one cent per pound.

"The large-scale curves from which were prepared the cost diagrams of the paper on 'The Possibilities in Bridge Construction by the Use of High Alloy Steels,' hereinafter referred to at length, afford a means of determining the economics of Mayarí steel for bridges as compared with nickel steels of 55,000 lbs. and 60,000 lbs. elastic limit. From them are found the following:

"500′ Simple Truss Spans

"Mayarí steel bridges at 1.5¢ per lb. excess over carbon steel are equal in cost to nickel steel bridges for $E = 55{,}000$ lbs. at an excess of 1.9¢ per lb., and to nickel steel bridges for $E = 60{,}000$ lbs. at an excess of 2.25¢ per lb. With Mayarí steel at 1.0¢ per lb. excess over carbon steel, the corresponding figures are, respectively, 1.35¢ and 1.7¢. For equal costs of bridges, as compared with carbon steel, Mayarí steel could stand an excess pound price of 2.1¢ for the manufactured superstructure.

"1000′ Simple Truss Spans

"With Mayarí steel at 1.5¢ per lb. excess, the excess for nickel steel of $E = 55{,}000$ lbs. is 2.25¢ per lb. and that for nickel steel of $E = 60{,}000$ lbs. is 3.0¢ per lb. With Mayarí steel at 1.0¢ per lb. excess over carbon steel, the corresponding figures are, respectively, 1.7¢ and 2.4¢ per lb. For equal costs of bridges, as compared with carbon steel, Mayarí steel could stand an excess pound price of 3.75¢ for the manufactured superstructure.

" Cantilever Bridges with Openings

from 1,000′ to 2,000′

"Mayarí steel bridges at 1.5¢ per lb. excess over carbon steel are equal in cost to nickel steel bridges for $E = 55{,}000$ lbs. at an excess of 2.3¢ per lb. and to nickel steel bridges for $E = 60{,}000$ lbs. at an excess of 3.1¢ per lb. With Mayarí steel at an excess of 1¢ per lb., the corresponding excesses for the other steels would be, respectively, 1.7¢ and 2.5¢ per lb. For equal costs of bridges, as compared with carbon steel, Mayarí steel could stand an excess pound price of 1.85¢.

"From the preceding it is evident that Mayarí steel has carbon steel beaten for bridgework under all conditions, but that if it costs when manufactured 1.5¢ per pound more than that metal, it will not be as economic as either of the grades of nickel bridge steel which can be produced commercially today. If, however, the manufacturers of Mayarí steel and of structures made therefrom can bring the price of their finished metal work down to an excess of one cent per pound as compared with carbon steel, their product will have somewhat more than a fighting chance in the competition. Nevertheless it will always have one serious obstacle to contend against, viz., the irregularity of the composition and characteristics of the finished product. This is shown very clearly in Mr. Reynders' letter; for in his shape-steel tests the elastic limit varies from 54,300 to 64,700 pounds per square inch, the ultimate strength from

91,400 to 98,420 pounds per square inch, the nickel content from 1.27 to 1.51 per cent, and the chromium content from 0.36 to 0.47 per cent. In the plate tests the corresponding variations were, respectively, from 55,270 to 64,660 pounds per square inch, from 90,040 to 99,600 pounds per square inch, from 1.36 to 1.57 per cent, and from 0.31 to 0.49 per cent. Considering that the raw material receives very little preparation for smelting, the preceding showing is by no means bad, especially since the records given indicate that no special difficulty has been experienced in complying with the specifications. On the other hand, though, the serious disadvantage under which the alloy labors is strikingly made evident by averaging the elastic limits given in the specimen tests; because the mean of all the figures is 59,655 lbs., while the requirement was only 50,000 lbs. It is possible that experience in the production of the alloy will result in greater regularity and less cost. If such prove to be the case, Mayarí steel is likely to supplant entirely the other alloy bridge steels at present obtainable; but it is far from being the ideal alloy for long-span bridge construction. Even if the inherent irregularity be made truly non-injurious to the metal by always keeping its characteristics well above the specified requirements, there will (for many years, at least) exist in the minds of purchasers the latent doubt of the steel's reliability and the dread that, without warning, the elastic limit and the ultimate strength may drop dangerously below the minima called for in the specifications.

"For a long time to come, and perhaps always, it will probably be necessary to test Mayarí steel much more thoroughly than carbon steel in order to prevent the utilization of any inferior melt or rolling in the manufacture of bridge superstructures.

"In the development of Mayarí steel for bridgework credit is due to Ralph Modjeski, Esq., C. E., the Consulting Engineer on the new Memphis Bridge, the first large structure in which that alloy is to be used.

"During a stay of some six weeks in France in 1909, the author learned that certain metal manufacturers in that country were making, in melts of five tons or less, by the electro-metallurgical process a purified steel for which they claimed rather astonishing results in respect to high elastic limit, great ultimate strength, and general suitability for the manufacture of bridges; although, as far as the author could ascertain, no such structures up to that time had been built of the new product. It was not convenient for him then to obtain and test specimens of the steel, as he greatly desired to do; hence he had to content himself with second-hand information obtained by both interviews and correspondence. The results of these convinced him that the claims made might, at least partially, be justified by performance; thereupon, having some spare time, he prepared an economic study of the possibilities for utilizing such purified steel in bridges. In his calculations he employed French units, prices, and other conditions, publishing the results in French in a memoir for

Le Génie Civil under the title, '*Étude Économique de l'Emploi de l'Acier au Carbone à Grande Résistance pour la Construction des Ponts.*'

"The French metallurgists, steel manufacturers, and bridge engineers to whom the author applied for information were all most kind and courteous in furnishing it, enabling him to collect quickly all the general data needed. Just here the author claims the privilege of expressing publicly his high appreciation of the exceeding kindliness and courtesy which French engineers and French scientists make a practice of showing toward their professional brethren from the United States. Nothing seems to give them too much trouble in their endeavor to oblige; and they are ever ready to devote hours of their valuable time to discussing the similarities and differences between French and American conditions, practice, and customs in all matters of a technical nature.

"The excess cost of the French purified steel, as compared with the ordinary carbon bridge steel of that country, appeared to be about nine-tenths of a cent per pound for the manufactured superstructure. The investigation showed the economics for its employment in bridge building for the mean and the extreme conditions of the French metal market, and for a number of assumed elastic limits, varying from 30 to 45 kg. per sq. mm., the value for the usual carbon bridge-steel in France being 24 and that for the author's specified nickel steel 42.5 kg. per sq. mm. The outcome of the investigation was that there was found no advantage whatsoever for the 30 kg. elastic limit; none for short spans, but a small one for long spans with a 35 kg. elastic limit; a decided saving for all cases with a 40 kg. limit; and a wonderful economy for the 45 kg. limit, the highest claimed by any of the French manufacturers.

"Figs. 4*h*, 4*i*, 4*j*, and 4*k* are taken from the issue of *Le Génie Civil* dated August 7, 1909. They show for carbon steel, the author's specified nickel steel, and the purified steels having assumed elastic limits of 30, 35, 40, and 45 kg. per sq. mm., respectively, the weights of metal in kilogrammes per lineal meter for simple-span bridges, ditto for cantilevers, the costs in francs per lineal metre of span of the steel erected in simple-span bridges, and the same in cantilever bridges of the most usual type. In Figs. 4*j* and 4*k* the assumed condition of the carbon steel market was that which existed in France at the time the investigation was made. Most fortunately, it was also the exact mean of the two extreme conditions.

"It was the author's hope that the publication of his paper would give an impetus in France (and perhaps elsewhere also) to the manufacture of bridges of purified steel, but the hope has proved to be a vain one; for, up to the present, he has not heard of any such development. It is probable that the metallurgists and the bridge manufacturers of France are no more eager to adopt drastic innovations in their practice than are their brethren in the United States. If nothing ever comes of that investigation, and if purified steel is never used directly for bridge building, it is within the realm of possibility that the ideal future alloy of steel

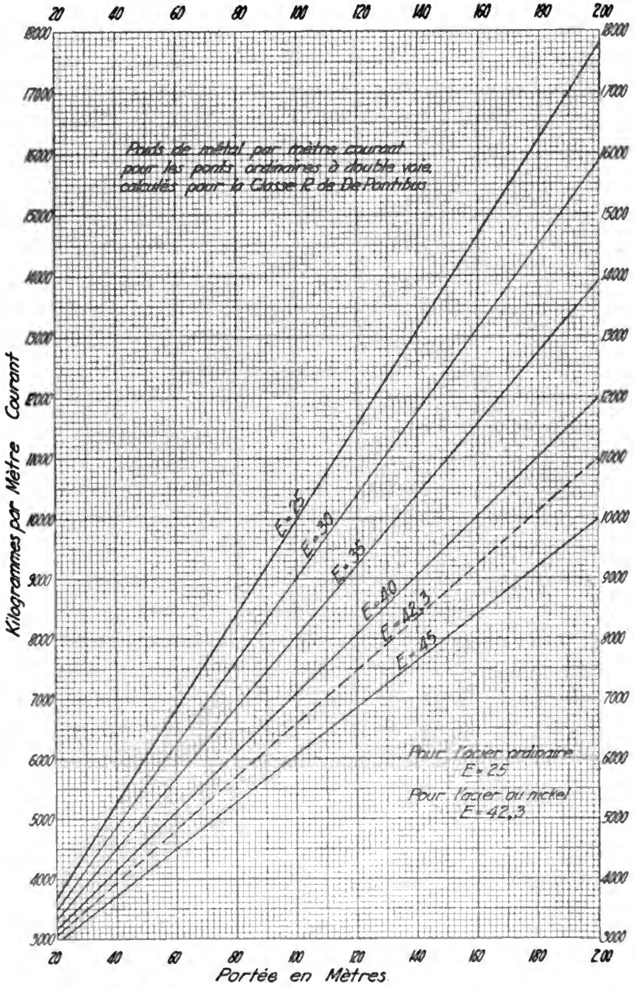


FIG. 4*h*. Poids de Métal par Mètre Courant pour les Ponts Ordinaires à Double Voie.

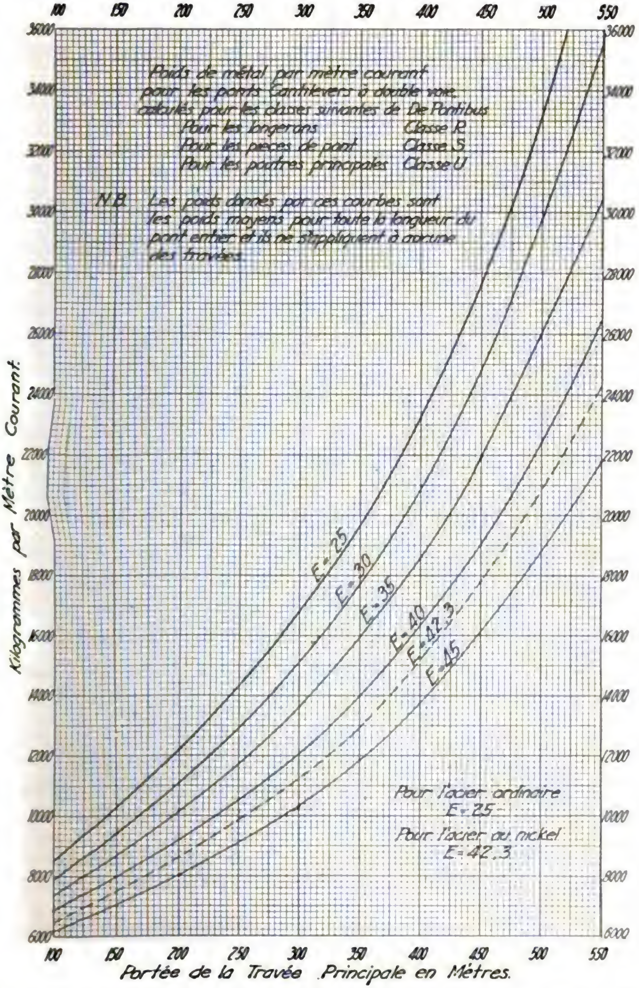


Fig. 4i. Poids de Métal par Mètre Courant pour les Ponts Cantilevers à Double Voie.

for bridges will be made by first purifying carbon steel, either by the electro-metallurgical process or by some other method, before the alloying element is added to the molten mass, in which case the trouble that the author went to in preparing the paper just described would not be wholly wasted.

"Desiring to obtain for the preparation of this paper some authentic information concerning the status of the manufacture of purified steel, the author consulted the United States Steel Corporation on the subject; and in reply to his letter received a communication from W. R. Walker, Esq., the Assistant to the President of the company, dated April 28, 1914, from which the following extract is quoted:

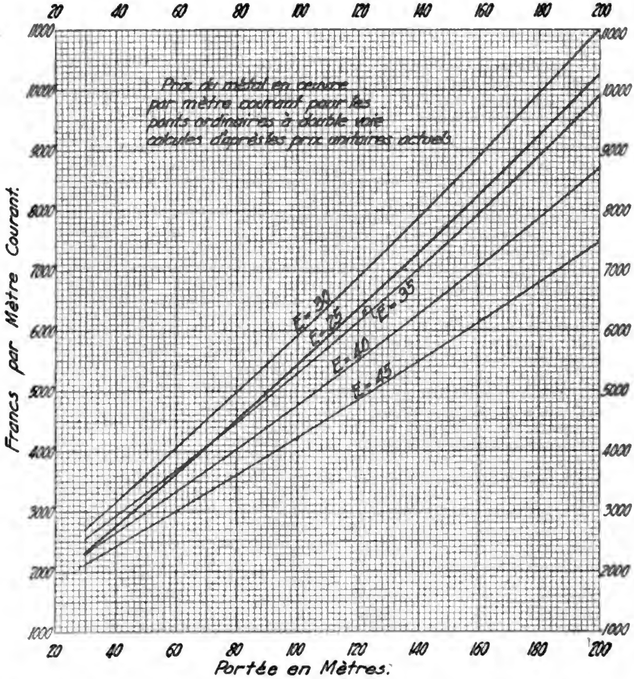


FIG. 4j. Prix du Métal en Œuvre par Mètre Courant pour les Ponts Ordinaires à Double Voie.

"'Although the electric steel process is a comparatively new one, being only ten years old, nevertheless there are in operation about 130 electric furnaces in this country and Europe which are making high-grade steel commercially. This steel has proven to be of such excellent quality that it is rapidly displacing crucible steel, which is generally considered the standard of excellence. It is also being used in seamless tubes, wire, sheets, ship angles, rails, thin armor plate, and especially tools.

"'In 1909 the Steel Corporation began the operation of a 13-ton electric furnace at the South Chicago plant of the Illinois Steel Company, and in 1910 began the operation of a similar furnace at the Worcester Works of the American Steel and Wire Company.

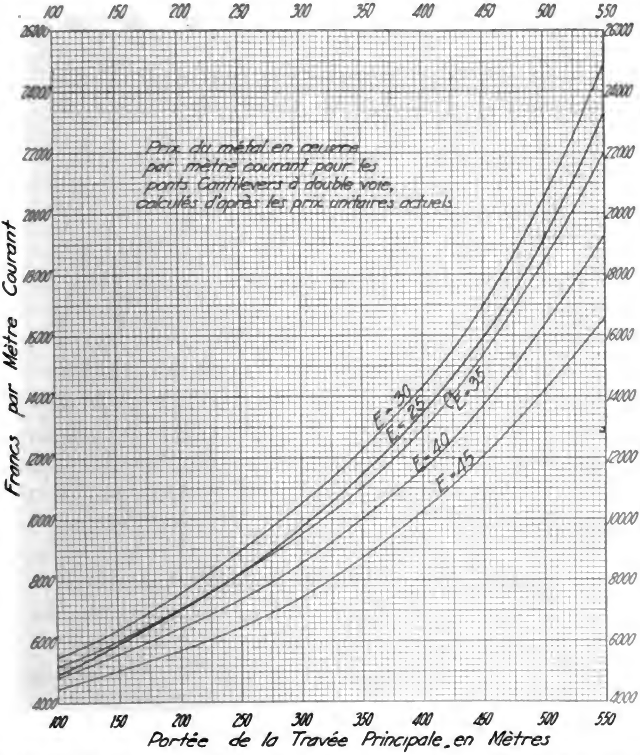


FIG. 4*k*. Prix du Métal en Œuvre par Mètre Courant pour les Ponts Cantilevers à Double Voie.

These installations were, at the time, experimental to the extent that it was not known if we could make the heavier products, such as rails, commercially. We have made about 10,000 tons of rails which are now in track. A number of years are required to test out the value of rails; but, up to this time, none of our electric steel rails have broken in service—even those located in the far Northwest during the very severe winter of 1912.

"'Although the electric furnace at Worcester has not been in operation for some time, due to commercial reasons, the furnace at South Chicago has operated almost continuously ever since its installation, and is now employed in making alloy steels, such as nickel-chrome, chrome-vanadium, high silicon steels, etc.

"'From these facts I think you can safely conclude that electric steel is no longer an experiment and that its quality is equal to the best made by any other process.'

"Late in 1913, not being satisfied with the progress then being made in the use of special steels of high elastic limit for long-span bridges, the author prepared for the American Society of Civil Engineers a paper entitled 'The Possibilities in Bridge Construction by the Use of High Alloy Steels.' It was printed in the Society's Proceedings early in 1914, and has since been discussed by some sixteen engineers of the United States,

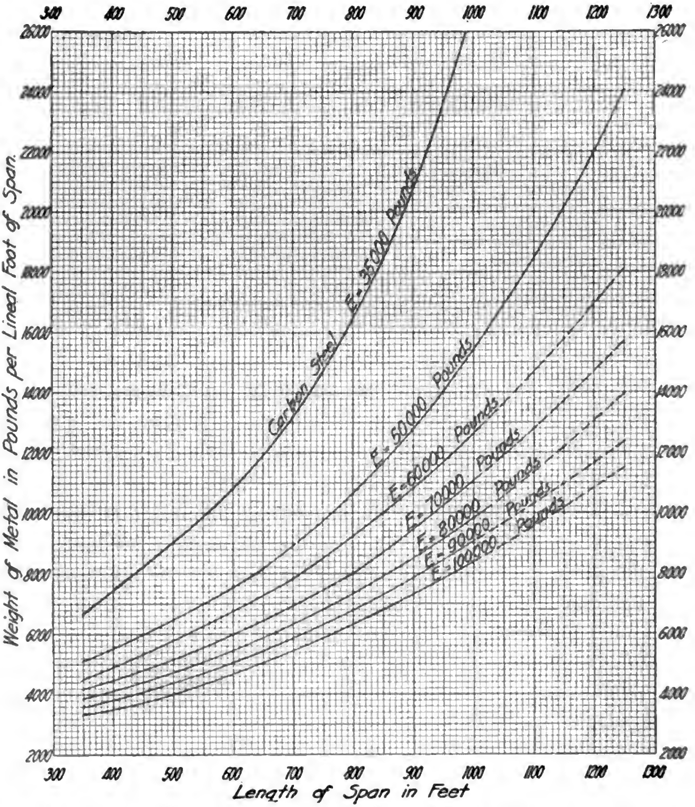


FIG. 4l. Total Weight of Metal per Lineal Foot of Span for Double-track, Simple-span Bridges of Carbon Steel and Alloy Steels of Different Elastic Limits.

Europe, and other parts of the world, and is now about to be published in the Society's Transactions.

"As might be anticipated from the title of the memoir, its object is to determine, for the usual types of bridges and for all practicable span lengths, the weights of metal per lineal foot of structure that would be required when using alloy steels of varying elastic limits, and the economics involved by their employment. Incidentally, there would be found the extreme practicable limits of span-length for cantilever bridges constructed for the greater part of such materials.

"The elastic limits assumed varied by 10,000 lbs., starting with 50,000 lbs. and ending with 100,000 lbs. Fig. 4*l* gives the weights of metal per lineal foot of span for double-track, simple-span bridges, and Fig. 4*m* records those for double-track, cantilever bridges. In the text of the memoir are given directions for finding the corresponding weights for similar bridges having more than two tracks and for those carrying other live loads than the ones assumed in the investigation.

"From Fig. 4*l* it is evident that in simple-span structures there is an immense saving in weight of metal by using alloy steel instead of carbon steel, also that the rate of saving diminishes gradually as the elastic limit of the metal increases.

"In Fig. 4*m* the saving of material by employing alloy steels, while not quite so striking as in the case of Fig. 4*l*, is still most apparent.

"If, as can be seen by Fig. 4*m* to be logical, it be assumed that a limit of 36,000 lbs. of metal per lineal foot of span is as high as it is either economical or practicable to go in the building of double-track, railway, cantilever bridges, the corresponding limiting lengths of main openings will be approximately as follows:

For carbon steel, E = 35,000 lbs..................2030 feet
" steel in which E = 50,000 " ....2340 "
" " " " E = 60,000 " ....2590 "
" " " " E = 70,000 " ....2780 "
" " " " E = 80,000 " ....2910 "
" " " " E = 90,000 " ....3030 "
" " " " E = 100,000 " ....3140 "

"From the appearance of the curves at their superior ends one may draw the conclusion that, in the case of the very-high-alloy steels, the limit of weight of metal per lineal foot of span can legitimately be raised beyond the previously assumed 36,000 lbs. The more nearly these curves approach the vertical the more uneconomical would it be to extend the limit beyond the said 36,000 lbs. per lineal foot.

"In studying the economics of the various alloy steels, the present ruling pound prices for carbon steel bridges erected were assumed to be 4.5 cents for simple spans and 5 cents for cantilevers.

"Fig. 4*n* is a specimen of the economic diagrams for simple-truss bridges. It shows that, even with an excess pound price of 4.5 cents

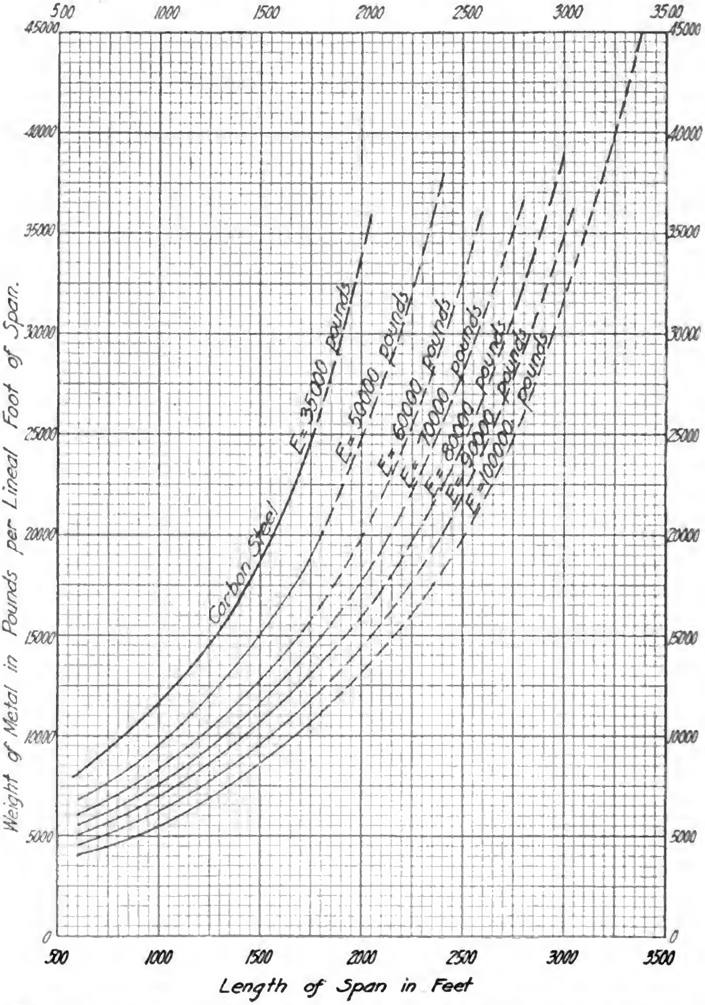


FIG. 4m. Total Weight of Metal per Lineal Foot of Span for Double-track, Cantilever Bridges of Carbon Steel and Alloy Steels of Different Elastic Limits.

for the fabricated alloy metal, there is a saving over carbon steel when the elastic limit of the alloy is 80,000 lbs. per square inch. Fig. 4*o*, which is for cantilevers and for an elastic limit of 80,000 lbs. per square inch, shows that the same conclusion holds as that just drawn for simple-truss spans.

"In concluding the memoir the author says it is evident that his results clearly prove that a systematic series of experiments made in search of a suitable and satisfactory alloy steel for building long-span bridges would be well worth while. He indicates that he is of the opinion that the first step to take in such an investigation would be to experiment on "purified" steel so as to bring it to its maximum of effectiveness, then to try adding nickel in various quantities, and afterward nickel with

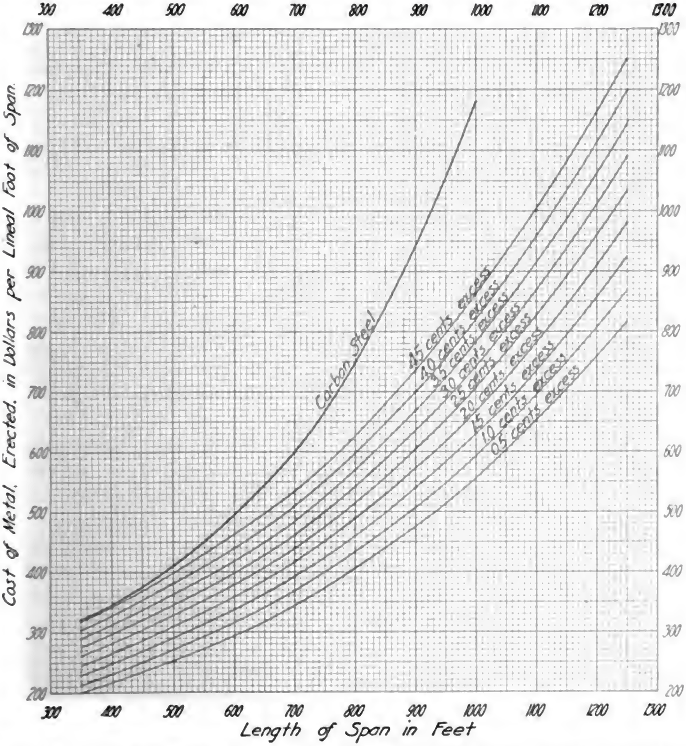


Fig. 4*n*. Comparative Costs of Double-track, Simple-span, Railway Bridges of Carbon Steel and Mixed Alloy and Carbon Steels for $E = 80{,}000$ lbs.

other but cheaper substances. He recognizes that augmenting the carbon in the purified steel, while increasing both its ultimate strength and its elastic limit, would tend to harden the metal, but anticipates that the addition of nickel (and possibly other elements) would tend to reduce the brittleness and render it workable.

"The final paragraph of his paper reads as follows:

"'The problem of finding a high, cheap alloy of steel, suitable in every particular for bridges, is now before the metallurgists and the builders of large metallic structures; and the values of all the results probably attainable are clearly indicated in this paper; hence the onus is on the engineering profession to see that the necessary experiments are arranged for and thoroughly carried out, in order that the world may have at its command a new metal that will permit of the spanning of waterways which are so wide and so deep, or are so restricted by navigation requirements, as at present to defy the art of the bridge engineer.'

"In the sixteen discussions of the paper there were advanced two pertinent suggestions concerning how to find the desired alloy. One was to use about three (3) per cent of aluminum as the principal alloy element, the present price thereof being only 20 cents per pound; and it was anticipated that such a combination might produce a satisfactory steel having an elastic limit of 100,000 lbs. per sq. in. It was evident from the way in which the discussion was worded that no experimenting worth mentioning had been done on that alloy for the purpose of bridge building; hence the suggestion must be treated as a wholly tentative one, although decidedly alluring.

"The other suggestion, made by Geo. L. Norris, Esq., Metallurgical Engineer of the American Vanadium Company of Pittsburg, was that either vanadium-carbon steel or vanadium-chromium steel be used as a high alloy for bridgework. This suggestion was much more directly to the point than the other; because both of the vanadium alloy steels mentioned have been manufactured for several years, although not for the purpose of bridge building. Mr. Norris was able to give the chemical and the physical qualities of the alloys recommended, but, unfortunately, only by stating very wide limits therefor.

"In answer to a long list of questions concerning the use of vanadium in steel, propounded by the author in a letter to the American Vanadium Company, and incorporated as a part of the résumé of discussions, the following important information was obtained from Mr. Norris:

"*A*. Vanadium steel is eminently fitted for the manufacture of eyebars. The elastic limit for full-size, chrome-nickel-vanadium bars varied in the tests from 63,280 lbs. to 80,480 lbs., and the ultimate strength from 93,000 lbs. to 99,800 lbs., the results depending upon the drawback or annealing temperature after quenching. The excess cost per pound of these finished bars after treatment, as compared with ordinary eyebars of carbon steel, Mr. Norris indicated would not exceed 3.5 cents.

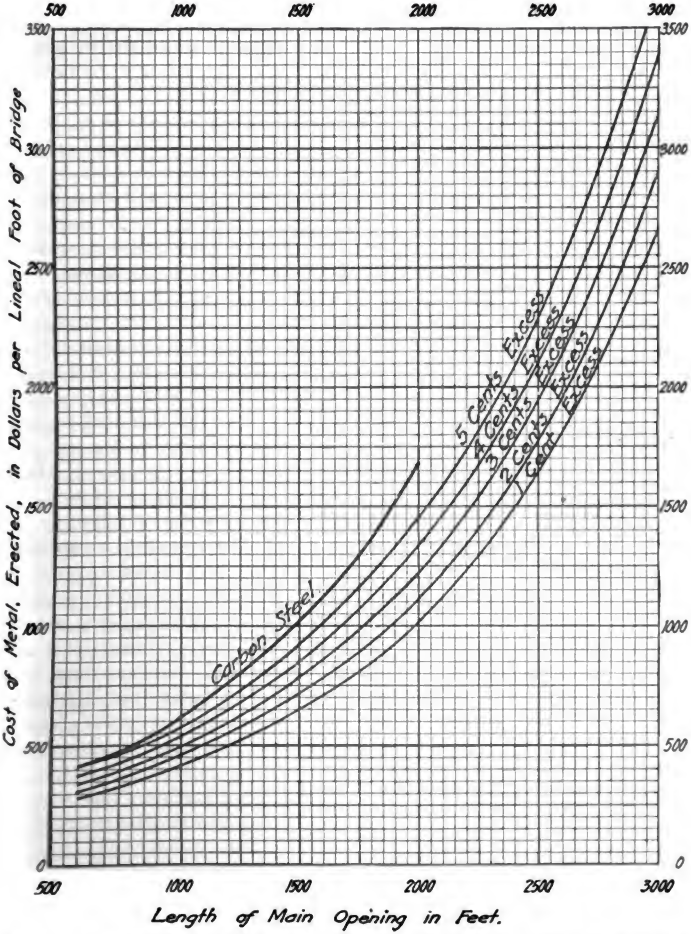


FIG. 4o. Comparative Costs of Double-track, Cantilever, Railway Bridges of Carbon Steel and Mixed Alloy and Carbon Steels for $E = 80{,}000$ lbs.

"He anticipated that simple carbon-vanadium steel eye-bars would have *E. L.* = from 60,000 to 75,000 lbs., and *Ult.* = from 85,000 to 100,000 lbs.; but it is evident that no experiments have been made on finished bars of that kind. The excess cost of such eye-bars after annealing, as compared with ordinary eye-bars of carbon steel, would not be more than 1.5 cents per pound.

"*B.* It was not quite so evident from the discussion that vanadium steel is as suitable for built members of bridges as it is for eye-bars, but it seems probable that it would be found satisfactory.

"Mr. Norris states that his chrome-vanadium steel will be workable under shop manipulations.

"*C.* It appears from Mr. Norris's remarks that heat-treated vanadium steel can be manufactured into built members of bridges without losing the great effect of the treatment, but it would probably be better to drill the rivet holes solid than to subpunch and ream them. Mr. Norris did not answer the question whether vanadium steel can be bent cold without injury; hence one may surmise that it cannot. However, this would not militate greatly against its use in bridge work, because in long-span structures (the only kind now under consideration) there should be very little, if any, metal to be bent.

"*D.* In respect to rivets, Mr. Norris advises, for chrome-vanadium steel, *E. L.* = 50,000 to 65,000 lbs., and *Ult.* = 70,000 to 90,000 lbs.; and for simple carbon-vanadium steel *E. L.* = 40,000 to 55,000 lbs., and *Ult.* = 65,000 to 85,000 lbs. The author is of the opinion that some serious difficulty might be encountered in cutting out defective rivets as high in strength as those first mentioned.

"Concerning rivets for bridges built of high alloy steels, it is a foregone conclusion that the ratio of the strength of the alloy-steel rivets to that of carbon steel rivets cannot be as great as the corresponding ratio of strength of plate-and-shape alloy steel to that of plate-and shape carbon steel. On this account, in high-alloy-steel bridges it will be necessary to use proportionately either more rivets or greater rivet diameters—or both.

"*E.* The amount of chromium recommended by Mr. Norris varies from 0.6% to 0.9% in combination with manganese varying from 0.4% to 0.6%, or even to 0.8%.

"*F.* Although for a number of years it was thought by the engineering profession in general that vanadium in steel acts merely as a scavenger, none of it remaining in the finished product, but all of it passing off with the slag, Mr. Norris asserts that about 80% of the vanadium which is added to the charge remains in the metal.

"Mr. Norris is positive that the vanadium is very evenly distributed through the ingot, and that not only is there no danger whatsoever of its segregation, but also that its presence in the molten metal tends to prevent the segregation of other substances—notably carbon. This is most reassuring and is a strong point in favor of the employment of vanadium steel for bridge building.

"*G.* From what Mr. Norris states, one might anticipate that the gain by adding nickel to vanadium-carbon steel or to vanadium-chromium steel would be either very small or non-existent.

"*H.* Mr. Norris is quite convinced that the addition of titanium to vanadium steel would be useless. Perhaps the explanation for this is that the 20% of the vanadium charge which passes off into the slag acts as a scavenger, thus obviating the necessity for any further purification —which office is the sole function of the titanium.

"The author's assumption from Mr. Norris's data of $E. L. = 60{,}000$ lbs. for his vanadium-carbon steel and $E. L. = 70{,}000$ lbs. for his vanadium-chromium steel with excess pound prices for the manufactured metals of 2.0 cents and 4.0 cents, respectively, would establish specifications and prices that the steel makers and bridge manufacturers should have no special trouble in living up to. It is possible that these assumptions do not do sufficient justice to the vanadium steels, but the figures had to be made safe for an investigation of the economics of the two types of vanadium steel compared with carbon steel and with the two classes of nickel steel which are procurable today in the United States for bridgework, viz., that for $E = 50{,}000$ lbs. at an excess pound price of 2.0 cents and that for $E = 55{,}000$ lbs. at an excess pound price of 2.5 cents.

"Under the preceding conditions the economics of the five steels considered are shown for simple-truss spans in Fig. 4*p* and for cantilever bridges in Fig. 4*q*.

"Referring to Fig. 4*p*, it is seen that for simple-span bridges, with the conditions assumed, vanadium-carbon steel shows, for all span-lengths, a small but material advantage over all the other steels; and that the vanadium-chromium steel begins to develop an economy over carbon steel at a span-length of about 500 feet, and over the two nickel steels at a span-length of about 650 feet.

"Referring to Fig. 4*q*, it is seen that for cantilever bridges, under the conditions assumed, vanadium-carbon steel shows for all span-lengths, as in the case of simple-truss spans, a small but material advantage over all the other steels; and that the vanadium-chromium steel begins to develop an economy over the carbon steel and the two nickel steels when the main opening has a length of about 1,400 feet.

"The author closes the résumé of the discussions of his paper on 'The Possibilities in Bridge Construction by the Use of High-Alloy Steels' in these words:

"'Summarizing the findings of the discussion and the résumé, there can be drawn the following conclusions:

"'First. Titanium as a scavenger of carbon steel promises good and useful results at exceedingly low cost. While it does not increase greatly the elastic limit or the ultimate strength of the metal, it makes it much more uniform and reliable. On that account it should be employed in a few cases on bridgework; and then, if it be found satisfactory, its adoption should be made obligatory by the railroad companies and the other builders of carbon steel bridges.

"'Second. There appears to be a great possibility in the use of aluminum as an alloy for bridge steel; but, as far as the author can determine, very few experiments in aluminum steels have yet been made; hence the said possibility is more or less hypothetical.

"'Third. The possibility of obtaining a good, high alloy steel for bridges by the use of vanadium appears to be almost a certainty; but the highest elastic limit and ultimate strength which can be obtained upon a commercial basis by the use of that element cannot be determined without making some elaborate and exhaustive experiments.'

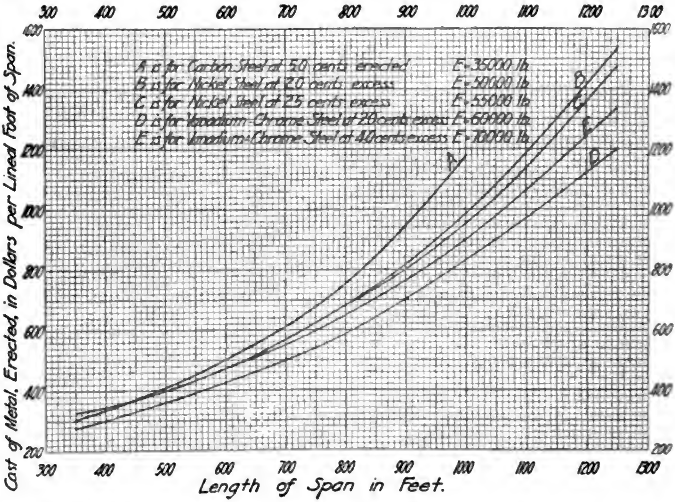


FIG. 4p. Comparative Costs of Double-track, Simple-span, Railway Bridges of Carbon Steel and Mixed Alloy and Carbon Steels, Contrasting Vanadium and Nickel Steels.

"In closing this memoir for the International Engineering Congress, the author feels that he ought to conclude with an apology to such an august assembly for the inherent personality of practically its entire substance—and such an apology is herewith tendered with all due respect. In extenuation of his transgression, however, he would state that, in relation to the subject of 'alloy-steels in bridgework,' he has been so closely

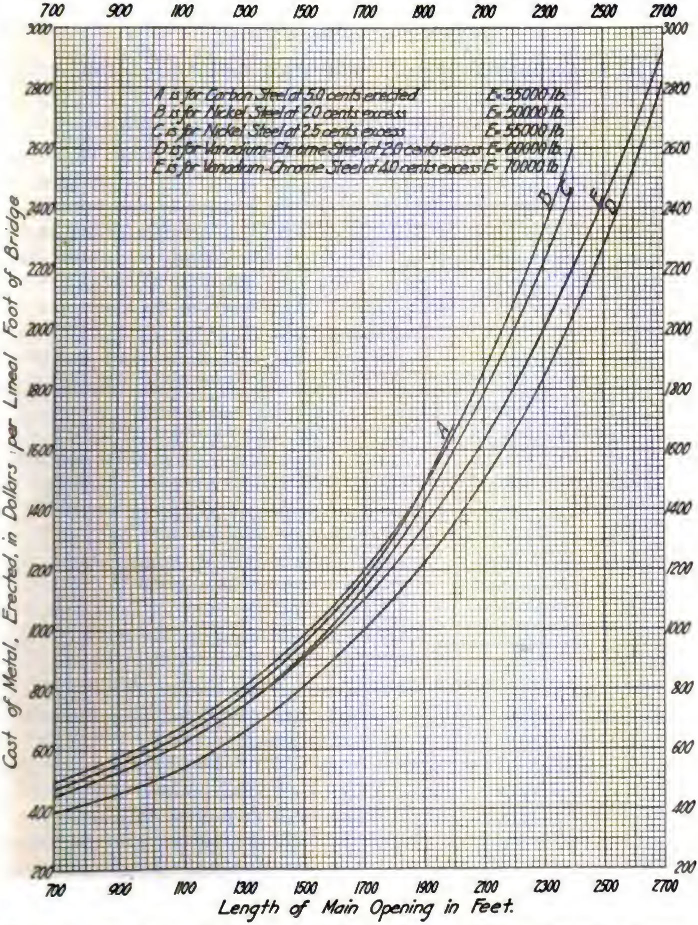


FIG. 4q. Comparative Costs of Double-track, Cantilever, Railway Bridges of Carbon Steel and Mixed Alloy and Carbon Steels, Contrasting Vanadium and Nickel Steels.

connected with the matter ever since its inception that he feels justified in applying to his case the words of the renowned Æneas,

'et quorum pars magna fui.'

"Addendum, Sept. 11, 1915

"Events in the development of alloy steels for bridgework move rapidly these days; and as it is probable that those members of the Congress who are at all interested in the subject of this paper would like to hear the latest word thereon, the following additional information is presented:

"Silicon Steel

"There is being used for the main built-members of the trusses of the C. B. & Q. R. R. bridge, now in process of construction, across the Ohio River at Metropolis, Ill., an alloy that has been termed 'silicon steel.' Its specified composition and characteristics are as follows:

Phosphorus, max. (basic)	0.04%
Phosphorus, max. (acid)	0.06%
Sulphur, max.	0.05%
Carbon, max.	0.40%
Manganese, max.	1.00%
Silicon, min.	0.25%
Ultimate Strength, min.	80,000 lbs.
Ultimate Strength, max.	95,000 lbs.
Yield Point, min.	45,000 lbs.
Percentage of Elongation in 8 inches = 1,600,000 ÷ Ultimate Strength	
Reduction of Area, min.	35%
Fracture desired	silky
Cold bend without fracture,	
¾ inch thick and under	d = t
¾ to 1½ inches thick	d = 1.5t
Over 1½ inches thick	d = 2.5t

"The excess cost of the rolled silicon steel, as compared with the ordinary carbon bridge steel, is one-half cent per pound, and the excess cost for shop work is about 0.15 cent per pound.

"The author is curious to know what would be the effect on the above steel if the twenty-five points of silicon were either omitted altogether or reduced to a minimum. His surmise is that very little difference would be noted; for, as stated in Chapter III, ordinary carbon steel containing forty points of carbon and about sixty-five points of manganese has a ultimate strength of 80,000 lbs., which would be increased somewhat by putting in thirty-five additional points of manganese.

"The high steel for bridges specified by the author nearly two decades ago in *De Pontibus*, but never used by him in any of his work, was as follows:

Phosphorus, max (acid)	0.07%
Sulphur, max.	0.05%

Silicon, max	0.06%
Manganese, max	0.80%
Ultimate Strength, min	70,000 lbs.
Ultimate Strength, max	80,000 lbs.
Yield Point, min	40,000 lbs.
Elongation in 8 inches	from 16 to 20%
Reduction of area	from 30 to 38%
Fracture desired	silky
Cold bend without fracture	d = 2t

"Such bridge metal is not greatly inferior to the silicon alloy just described; and if the permissible manganese were increased from the specified 80 to the 100 points permitted in silicon steel, the differences in resistance characteristics of the two steels would be somewhat reduced. Again, the specifications for the silicon steel require all rivet holes to be drilled from the solid, an old requirement for high carbon bridge steel.

"From what precedes, the author is led to the conclusion that the 'silicon steel' of the Metropolis Bridge is merely a slightly improved form of high carbon steel, notwithstanding the fact that his friend, C. W. Bryan, Esq., C. E., Chief Engineer of the American Bridge Company, who furnished him with all of the preceding data that are not given in *Engineering News* of July 29, 1915, is of the opinion that silicon steel suitable for bridgework may be obtained later on with greater elastic limit than 45,000 lbs. per square inch.

"From a commercial point of view silicon steel with $E = 45{,}000$, and at an excess pound price, compared with carbon steel, of one-half cent for the rolled raw material, is a close competitor of Mayarí steel with $E = 50{,}000$, and at a corresponding excess pound price of one cent. To prove the correctness of this statement, let us compare the costs of Mayarí steel and silicon steel, long-span, simple-truss, double-track bridges, erected, when the corresponding carbon steel bridges in place are worth 4.5 cents per pound. The weights of metal for an assumed span length of 750 feet, taken from Fig. 4*l*, are, respectively, 9,650 lbs. and 10,350 lbs. Neglecting, for convenience, the small variation of cost per pound involved in the erection, and assuming, as in the body of the paper, a total excess of 1.5 cents per lb. delivered at bridge site for the Mayarí steel, we have the following comparison:

Mayarí Steel, 9,650 lbs. at 6¢ = $579.00
Silicon Steel, 10,350 lbs. at 5.5¢ = 569.25

Difference in favor of Silicon Steel = $9.75 per lineal foot of span, or about 1.7 per cent.

"Vanadium Carbon Steel

"Since the time when Mr. Norris furnished the information concerning vanadium steel that is contained in the body of this paper, he has had made two melts of vanadium carbon steel which are of interest to bridge engineers, although the metal was intended for other purposes than bridge building. The results of these tests, which are by no means conclusive, and which, in their incompleteness, may properly be deemed somewhat disappointing, nevertheless hold a promise of the eventual attainment of the desired alloy. But to accomplish this, future tests must be made systematically and in accordance with a programme based on a thorough study of the problem by an expert bridge engineer who has in mind the needs of the advancing art of bridge construction and the possibilities awaiting it, if a high-elastic-limit alloy-steel can be had. If such an investigation were made, it might be practicable to discover a workable and perfectly satisfactory steel alloy of vanadium (and perhaps other special elements) for the manufacture of bridges. With a report from a recognized and disinterested authority, the engineering profession would be quick to see and avail itself of the advantages offered by this superior bridge metal. The author would be content, if there were obtained an alloy, suitable in every particular for bridge construction, having an elastic limit of 80,000 lbs. per square inch for annealed eye-bars of the largest size, and one of 70,000 lbs. per square inch for plate-and-shape steel; but it might be within the realm of possibility to raise the last figures as much as 5,000 lbs., especially as solid drilling can now be adopted instead of subpunching and reaming without adding materially to the cost of the shopwork. The author is of the opinion that such results can be obtained *by means of vanadium*, but only, as just mentioned, after a thorough and exhaustive investigation shall have been made by an expert bridge engineer. Such a person should be allowed ample funds for all legitimate expenditures involved in the exact determination of the different properties of a number of trial alloys of various compositions. Only in this way can the economics of the problem be settled. A picayunish policy adopted in the making of any important economic investigation is more than likely to defeat its purpose, and it is absolutely certain to involve a greater or less falling short of ultimate possible success in the complete attainment of the desired information."

After the résumé of the paper on "The Possibilities in Bridge Construction by the Use of High Alloy Steels" had gone to press, there was received from Dr. J. W. Richards, who for twenty years has been the Consulting Metallurgist of the Aluminum Company of America, the following communication:

"The statements of L. J. LeConte concerning the properties of aluminum steel do not agree with the rather extensive investigations of Sir Robert Hadfield, published in the Journal of the Iron and Steel Institute for 1890, Vol. II, page 161. In that paper and its discussion it was shown that additions of 2¼ per cent aluminum to 0.22 carbon steel had very little effect on the elastic limit or breaking strength, but reduced greatly the elongation and reduction of area, *i.e.*, it makes the metal more brittle.

"The paper concludes by recommending the advisability of adding not over 0.10 or 0.15 per cent of aluminum to steel, with the object, not of making an alloy, but that the addition may be consumed in deoxidizing the steel. This is, in fact, the proper and valuable function of adding aluminum to steel. Up to 0.1 per cent is added to Bessemer steel, 0.01 to 0.05 per cent to open-hearth steel, and 0.001 to 0.005 per cent to crucible or electric furnace steel. This may be called universal practice in the steel industry, for in practically every large steel works such minute additions of aluminum are now made, to ensure complete deoxidation and produce sounder castings."

This adverse opinion from such a high authority as Dr. Richards reduces very nearly to zero the chance of finding by the use of aluminum the desired high-alloy steel for bridgework, notwithstanding the fact that Mr. LeConte is most enthusiastic in his prognostications concerning the success which would attend a thorough series of tests of that alloy, as is evidenced by a late letter of his to the author.

The following table gives a record of tests, furnished through the courtesy of Henry W. Hodge, Esq., Consulting Engineer, of the nickel steel which he used in his St. Louis Free Bridge. At the right of it are appended the averages for all the elastic limits, ultimate strengths, elongations, reductions of area, and chemical compositions of the various specimens and full-size bars tested:

TABLE 4*d*

SPECIFICATIONS FOR NICKEL STEEL AND AVERAGE TESTS OF SPECIMENS THEREOF

Character of Test Piece, Etc.	Attribute or Constituent	Specified by Mr. Hodge	Specified by Author in Ni. St. for Br.	Average of All Tests
Unannealed Specimens	Elastic Lim.	55,000 lbs.	60,000 lbs.	60,250 lbs.
	Ultimate	95,000 lbs. to 110,000 lbs.	105,000 lbs. to 120,000 lbs.	99,850 lbs.
	Elong. in 8"	16%	15%	17.9%
	Red. of Area	25%	—	33.3%
Annealed Specimens	Elastic Lim.	52,000 lbs	—	56,250 lbs.
	Ultimate	90,000 to 105,000 lbs.	—	91,960 lbs.
	Elong. in 8"	20%	—	22.6%
	Red. of Area	35%	—	41.8%
Full-size Bars	Elastic Lim.	48,000 lbs.	49,500 to 57,750 lbs.	55,890 lbs.
	Ultimate	85,000 to 100,000 lbs.	90,000 to 105,000 lbs.	91,020 lbs.
	Red. of Area	35%	—	36.2%
	Elong. in 18'	10%	10% (in 10')	14.4%
Chemical Composition	Carbon	0.45% max.	0.42% max.	0.38%
	Phosphorus	0.04% max.	0.03% max.	0.012%
	Sulphur	0.04% max.	0.04% max.	0.03%
	Manganese	0.7 % max.	0.75% max.	0.58%
	Nickel	3.25% av.	3.50% av.	3.45%

The preceding table will give a means of comparing the characteristics of the nickel steel which manufacturers are willing to furnish with those of the alloy as specified by the author in "Nickel Steel for Bridges."

In respect to the Elastic Limit (the most important characteristic of all, in that it determines the intensities of working stresses,) Mr. Hodge specified 55,000 lbs. per square inch as a minimum for unannealed specimens, as against the author's 60,000, while the tests showed an average of 60,250, with 63,940 as the highest record and 53,580 as the lowest. In only one case did the elastic limit fall below Mr. Hodge's minimum of 55,000 lbs.

In respect to the ultimate strength of unannealed specimens, Mr. Hodge specified 95,000 lbs. as a minimum and 110,000 lbs. as a maximum, as against the author's 105,000 and 120,000 lbs. respectively, while the tests showed an average of 99,850 lbs., with 109,150 as the highest record and 91,600 as the lowest. In four cases only did the ultimate strength fall below Mr. Hodge's minimum of 95,000 lbs.

In respect to the elongation of test specimens in 8 inches, Mr. Hodge specified 16%, as against the author's 15%, while the tests showed an average of 17.9%, with 24% as the highest record and 10% as the lowest. In four cases only did the elongation fall below Mr. Hodge's specified minimum of 16%, and in one case only below the author's minimum of 15%.

In respect to the percentage of carbon, in no case did the quantity used exceed either the maximum of 0.45% specified by Mr. Hodge or that of 0.42% specified by the author. The average for all the tests was 0.38%—the exact average requirement given in the author's specifications.

In respect to the percentage of phosphorus, in no case did the quantity recorded exceed either Mr. Hodge's limit of 0.04% or the author's of 0.03%, the average being only 0.012%, showing that the steel was well purified from this most objectionable ingredient.

In respect to the percentage of sulphur, in no case did the quantity recorded exceed the 0.04% specified by both Mr. Hodge and the author, the average being 0.03%, indicating that the steel was satisfactorily free from this impurity also.

In respect to the percentage of manganese, the records show that the amounts used were always below the 0.7% specified by Mr. Hodge and the 0.75 specified by the author, the average being 0.58%. It is probable that the use of ten or fifteen more points of manganese would have strengthened the steel materially and raised its elastic limit several thousand pounds without inducing any objectionable characteristics.

In respect to the percentage of nickel, in no case did the record show less than the 3.25% minimum, specified by both Mr. Hodge and the author, or exceed the latter's specified maximum of 3.75%.

In respect to the elastic limit for full-size eye-bars, in no case did it fall below the limit of 48,000 lbs. per square inch specified by Mr. Hodge; but in eight instances it failed to come up to the author's specified requirement, which, by the way, varies inversely with the thickness of the bar.

In respect to the ultimate strength of full-size eye-bars, in three instances it was less than that specified by Mr. Hodge; and in most cases it was well below the author's specification. The average thickness of all the bars is $1\frac{29}{32}''$, for which the latter specification would call for an ultimate strength of about 96,000, while the average is only 91,000 lbs.

With the exception of one bar which broke in the head, all the bars complied easily with Mr. Hodge's specification for elongation.

It is evident by the preceding deductions from the table that the nickel steel furnished for the Free Bridge at St. Louis failed to come up to the specifications given in "Nickel Steel for Bridges"; and it will be interesting to surmise as to what should have been done to it in order to make it comply therewith.

First: As to Plate-and-shape Steel, the amount of nickel employed was just right, as was also the amount of carbon; and the percentages of the impurities, phosphorus and sulphur, were kept down properly; but the average for manganese was twelve points too low. The average elastic limit was only 250 lbs. above the author's specified minimum, and the average ultimate strength was some 5,000 lbs. below the same. While it is probable that the addition of twelve points of manganese would have brought the elastic limit nearly, if not quite, up to the author's requirements, it is not likely that it would have done so for the ultimate strength; hence some additional treatment would have been necessary. Possibly the author's requirement for minimum ultimate strength is too severe. It makes the ratio of elastic limit to ultimate 0.57, while Mr. Hodge's corresponding ratio is 0.58, which coincidence would tend to show that the trouble does not lie in that requirement. Probably its real cause is too great a variation in the values of both the elastic limit and the ultimate—especially the latter. To correct this objection the addition of a small amount of titanium to the molten charge would likely be successful. Its use would probably increase both the average elastic limit and the ultimate a few thousand pounds each and make the product much more uniform. The experiment is well worth trying.

Second: As to Eye-bar Steel, the amount of nickel employed falls short of the author's requirement by 0.8%, the amount of carbon by 0.07%, and the amount of manganese by 0.22%. If these deficiencies were made up, it is more than likely that the elastic limit and the ultimate strength of full-size eye-bars would readily comply with the author's specifications—especially if a little titanium were added to make the metal uniform.

Concerning the beneficial effect of the addition of titanium to steel, the reader is referred to the discussion by Monsieur Petinot of the author's before-mentioned paper, "The Possibilities in Bridge Construction by the Use of High Alloy Steels," published in the 1915 Transactions of the American Society of Civil Engineers.

CHAPTER V

DEAD LOADS

The dead load for any bridge usually consists of the weights of those parts of the permanent structure, which from their position cause stresses on the trusses or girders; but sometimes an extraneous dead load is allowed for, such, for instance, as snow or an accumulation of dirt, or a line or lines of pipe for water or gas, or an extra thickness of floor plank, or a possible future pavement. All parts of the structure resting directly upon or over the piers are to be omitted when figuring the dead load, such, for instance, as pedestals, end floor-beams, end bents in deck structures, and the portal bracing when the end posts are made vertical instead of inclined.

In all simple span trusses the dead load is assumed to be uniformly distributed over the entire length of span, and this is approximately correct; for the chords are heavy at the middle and light near the ends, while the web is the reverse. When the chords are parallel the assumption of uniform distribution is almost exact, but in long spans when the top chords are polygonal the web is not much heavier at the ends than it is at the middle; but, on the other hand, the chords are more nearly uniform in weight from end to end, and the lateral system becomes heavier as the ends of the span are approached. On the whole, this assumption of uniform distribution of dead load for simple truss spans is exact enough for all practical purposes.

But in the case of cantilevers, arches, long swing-spans, bascule bridges, and some other unusual types of structures, the dead load is not uniformly distributed over the entire span, and an assumption that it is would result in errors too large to be permitted. In such cases it is first necessary to assume the various panel dead loads, then design the bridge and compute them. If the resulting agreement is fairly close, well and good; but, if not, new dead loads are to be assumed, the corresponding stresses are to be figured, the proportioning of parts is to be done anew, and the resulting dead loads are to be computed. The second set of calculations should give a very close agreement; but, if not, the work must be gone through again with a third assumption of dead load distribution. In case of large or complicated structures it is best in figuring dead load stresses to adopt the unit load method described in Chapter X in order to facilitate the recalculation.

In ordinary spans it is customary to assume that two-thirds (⅔) of the dead load are concentrated at the panel points of the lower chords in through-bridges and at those of the upper chords in deck bridges; and one-third (⅓) of the dead load at the panel points of the upper chords in through-bridges and at those of the lower chords in deck bridges. If a bridge carry a very heavy, paved floor, it would be better to make the division three-quarters (¾) and one-quarter (¼); and in some extremely long spans it might be better to assume sixty (60) and forty (40) per cent; but this division of the dead load is not an important matter, as it affects generally only the vertical posts, which often have an excess of section; consequently any error in truss stresses caused by an improper division of dead loads above and below need cause no one any uneasiness.

The rule given in the specifications of Chapter LXXVIII is that after a span has been figured and the assumed dead load has been checked by the weights obtained from the diagrams of stresses and the computations of details, if there prove to be a variation exceeding one per cent of the sum of the equivalent live load, the impact load, and the actual dead load, the stresses and the proportioning of parts are to be calculated again with a new assumed dead load. It is better that the assumed dead load should be too high rather than too low, as such an excess tends to increase all stresses except those in counters; and in modern bridge designing the latter are confined to cheap highway spans. If an extra large dead load is assumed so as to provide for future possibilities, the minimum instead of the greatest possible dead load should be employed in figuring counter stresses, since the effect of the dead load is to reduce these.

The following are the unit weights of all the materials that ordinarily enter into the construction of bridges:

Creosoted lumber, from four and one half (4½) to five (5) pounds per foot board measure. Oak and other hard woods, excepting those from Australia and New Zealand, four and a quarter (4¼) pounds per foot board measure. Some varieties of Australian timber, used occasionally in bridge floors on the Pacific Coast, six (6) pounds per foot board measure. Yellow pine, three and three-quarters (3¾) pounds per foot board measure. White pine and other soft woods, two and three-quarters (2¾) pounds per foot board measure.

Rails and their fastenings for first-class, standard-gauge railroads (both steam and electric), about seventy (70) pounds per lineal foot per track. If the rails adopted be unusually heavy or unusually light, their exact weight (including fastenings) per lineal foot per track should be figured and used when computing the dead load.

Concrete, from one hundred and forty (140) to one hundred and sixty (160) pounds per cubic foot, according to the character of the stone or gravel used in its manufacture. For reinforced concrete five (5) pounds are to be added to the preceding unit weights.

Asphalt pavement, including binder, one hundred and twenty (120) pounds per cubic foot.

Brick pavement, one hundred and forty (140) pounds per cubic foot.

Steel, four hundred and ninety (490) pounds per cubic foot.

Earth (used as a covering for masonry or concrete arches) one hundred (100) pounds per cubic foot.

Broken stone for ballasted floors, one hundred (100) pounds per cubic foot.

Snow, compacted, fifty (50) pounds per cubic foot.

Water (carried in pipes), sixty-two and a half (62.5) pounds per cubic foot.

In computing dead loads for all ordinary bridges the weights of metal given in the diagrams of Chapter LV will be found of great assistance. In the case of cantilever bridges, the probable weight of metal and its distribution can be ascertained from Chapter XXV as well as from the diagrams of Chapter LV.

In assuming dead loads for arches, the information given in Chapter XXVI will enable one to estimate pretty closely to the actual dead loads, but the computer is advised to study the question very carefully, as the weight of metal in any arch bridge will depend considerably upon the number of panels, the ratio of rise to span, and the general type of structure.

In suspension bridges it will be necessary to know the weight per lineal foot for steel cables of various diameters. While these vary slightly in different makes of rope, those given in the following table for twisted cables will be found sufficiently accurate for all practical purposes.

TABLE 5*a*

WEIGHTS OF STEEL CABLES

Diameter of Rope	Weight of Rope in Pounds per Lineal Foot
1″	1.70
1¼″	2.65
1½″	3.82
1¾″	5.20
2″	6.80
2¼″	8.60
2½″	10.62
2¾″	12.85
3″	15.30

For intermediate diameters the weights may either be interpolated or found by the formula,

$$W = 1.7D^2,$$

where W is the weight of rope in pounds per lineal foot and D is its diameter in inches.

In a cable composed of straight wires, if n is the number of wires, d the diameter of each wire, and D the diameter of the rope,

$$n = 0.77\left(\frac{D}{d}\right)^2;$$

and if w be the weight per lineal foot for a single wire and W the weight per lineal foot of the rope,

$$W = nw = 0.77w\left(\frac{D}{d}\right)^2.$$

There is another loading in the nature of a dead load that must not be omitted from consideration, viz., the uplift loading at the ends of swing spans, producing reverse dead load stresses. This is treated at length in Chapter LXXVIII, to which the reader is referred. The amounts of uplift there given are as great as the usual machinery is capable of handling—possibly sometimes greater—for in a well-designed bridge the assumed height of the uplift at either end of span should be made a trifle in excess of the rise caused by the greatest possible live load, with its impact included, that can come upon the other arm. The reverse dead load stresses throughout each arm, due to uplift, should be considered only when they tend to augment the section of the piece; because, although it is true that they act in conjunction with the live load, it is possible that they may not always exist—at least not to the extent of the amounts computed.

CHAPTER VI

LIVE LOADS

In the early days of railway bridge designing the live load adopted was a simple uniform advancing load, amounting to about two thousand (2,000) pounds per lineal foot. This was soon increased to a long ton or two thousand two hundred and forty (2240) pounds per lineal foot. The next step was to place a locomotive at the head of the train, giving the spacing of the various axles and the loads upon them; and as it became customary to use double headers to haul long trains, the bridge loadings were soon increased by providing for two engines in advance of the cars.

Theodore Cooper, Esq., C. E., was one of the first engineers to establish standard live loads for railway bridges. He had in his first bridge specifications three classes thereof, as follows:

Class A. Two consolidation engines, each with its tender being about fifty (50) feet long and weighing 85.5 tons, followed by a train of cars weighing 3,000 lbs. per lineal foot, the axle loads being as follows:

Pilot	15,000 lbs.
Drivers, each	24,000 lbs.
Tenders, each	15,000 lbs.

Class B. Two consolidation engines of like length and wheel spacing, each with its tender weighing 80.5 tons, followed by cars weighing 2,240 lbs. per lineal foot, the axle loads being as follows:

Pilot	15,000 lbs.
Drivers, each	22,000 lbs.
Tenders, each	14,500 lbs.

Class C. Two mogul engines, each with its tender being about forty-seven (47) feet long and weighing 71.5 tons, followed by cars weighing 2,000 lbs. per lineal foot, the greatest axle loads being as follows:

Pilot	15,000 lbs.
Drivers	25,000 lbs.
Tenders	13,500 lbs.

These live loads of Mr. Cooper's have been gradually increased from time to time, until, in 1906, his heaviest engine and tender weighed 177.5 tons, the corresponding car load being 5,000 lbs. per lineal foot and the axle concentrations as follows:

Pilot	25,000 lbs.
Drivers	50,000 lbs.
Tenders	32,500 lbs.

The example set by Mr. Cooper was quickly followed by many of the railroad companies, which established standard live loads of their own, almost no two being alike, thus causing infinite labor and trouble for the computers of the bridge companies. This, however, was by no means the limit of their imposition, for soon the standard specifications called for the computations of stresses for two alternative loadings, then three, four, and in some extreme case five slightly different live loads. The figuring that such specifications necessitated was enormous, and, moreover, it was useless; for a single live load would have answered the purpose just as well. As the railroad engineers who wrote the specifications very seldom did any of the computing themselves, they did not recognize the extent to which they were imposing absolutely unnecessary labor upon the engineering profession, or else they were indifferent; for not only did they require every bridge member to be figured for each engine load, but they also insisted that all calculations should be made by the so called "exact method" of axle concentrations; of which more will be said in Chapter X. As time went on the railroad companies kept increasing the weights of their locomotives and trains, and the standard bridge specifications had constantly to have their live loads modified to keep pace with the increase.

In 1891, 1892, and 1893 the author made an extended investigation of the railway live load question by a systematic correspondence with the chief engineers of all the principal railroads of the United States, Canada, and Mexico, obtaining a consensus of opinion by several letter ballots. The results of his investigations were published in the Transactions of the American Society of Civil Engineers and in the technical press; and they evoked considerable discussion in the latter. All of these writings and discussions have been collected by Mr. Harrington and published in his book, "Principal Professional Papers." Even while this investigation was being made during the short period of two years, it was found necessary to advance the live loads, as can be seen by comparing the plates opposite pages 272 and 458 of Mr. Harrington's book. From these it will be noticed that the tender axle loads were increased three thousand (3,000) pounds each, and another standard loading, viz., Class T, was added.

The result of the two years' investigating and balloting was the establishment of what was termed "The Compromise Standard System of Live Loads for Railway Bridges," which is reproduced in Fig. 6*a*. An inspection of this will show that for the lightest loading, Class Z, the weight of one engine and tender was ninety-three and one-half (93.5) tons and the car load was three thousand (3,000) pounds per lineal foot, the axle loads being fifteen thousand (15,000) pounds for the pilot, twenty five thousand (25,000) pounds for the drivers, and eighteen thousand (18,000) pounds for the tenders. For the heaviest loading, Class T, the weight of one engine and tender was one hundred and forty-four and one-half (144.5) tons, and the car load was four thousand two hundred (4,200) pounds per lineal foot, the axle loads being twenty-one thousand

(21,000) pounds for the pilot, forty-three thousand (43,000) pounds for the drivers, and twenty-four thousand (24,000) pounds for the tenders. There were also alternative loadings for short spans consisting of two heavy axle loads spaced seven (7) feet centres, and varying from forty thousand (40,000) pounds each for Class Z to fifty-two thousand (52,000) pounds each for Class T. These live loads were incorporated in the first edition of *De Pontibus* (1898) and sufficed until 1900, when the author issued his book, "Specifications for Steel Bridges." In this he increased by regular gradations of axle loads the number of standard loadings by adding Classes S, R, and Q, the total weight of one engine and tender for the latter being one hundred and seventy (170) tons, the car load four thousand eight hundred (4,800) pounds per lineal foot, the pilot axle load twenty-four thousand (24,000) pounds, the driver axle load fifty-two thousand (52,000) pounds, the tender axle load twenty-seven thousand (27,000) pounds, and the alternative axle load fifty-eight thousand (58,000) pounds. These heavier standard loadings were incorporated in the second edition of *De Pontibus*, which was issued in 1903.

About this time, though, certain railroad companies in order to make long runs, especially through the arid portions of the Western States, increased the capacities of their tenders for both coal and water, and, in consequence, issued new live load diagrams differing from both those of the Compromise Standard and the standards established by Mr. Cooper in his various bridge specifications. The latter, by the way, differed but slightly from those of the Compromise Standard, the principal variation being that Mr. Cooper's axle loads were all multiples for the different classes, while in the Compromise Standard the axle loads varied by constant increments. The multiple feature is a slight convenience, mainly in computing the equivalent uniform loads and the total end shears. The reason that it was not adopted for the Compromise Standard was that it would not fit in with the averages deduced from the ballots voted by the chief engineers of the various railroads.

Of late years the tendency has been to increase materially the weights of engines, tenders, and cars, and it was not very long before certain railroads began to specify live loads even heavier than Waddell's Class Q. There was one noticeable feature in their loadings, however, viz., that the total lengths of locomotives and tenders were increased, thus reducing the average loads per lineal foot and making the effective increase in live loadings more apparent than real.

In 1904 at the annual convention of the American Society of Civil Engineers, Henry W. Hodge, Esq., C. E., read a paper on "Live Loads for Railroad Bridges"; and it was largely discussed both at the meeting and subsequently. The paper and the discussions are printed in Part A, Vol. LIV, of the Society's *Transactions* for 1905. Mr. Hodge recommended that any road which expects to do an ordinary traffic, or to carry the freight delivered to it by other large systems, should not use an engine load having less than fifty thousand (50,000) pounds on drivers, followed by a car load not less than five thousand (5,000) pounds per lineal

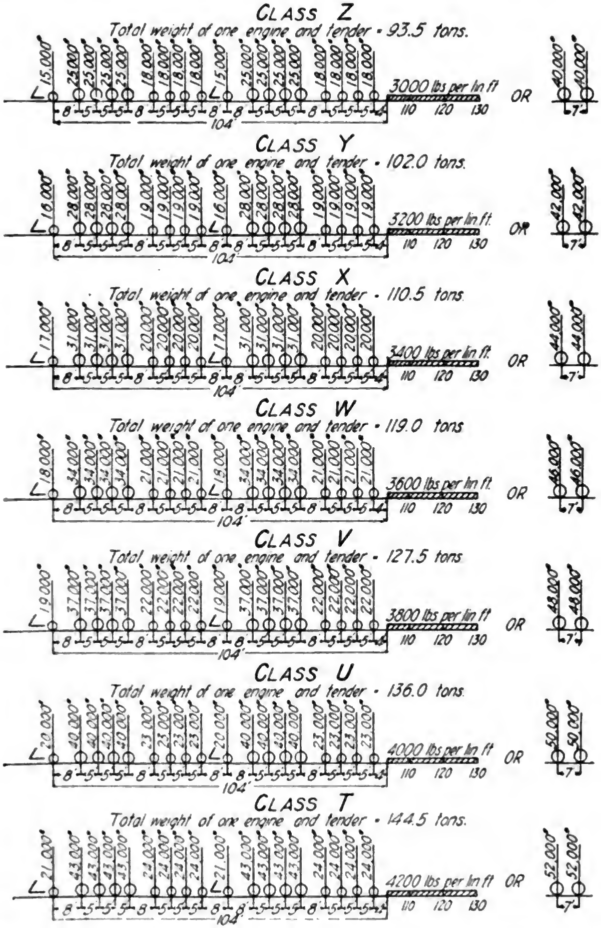


FIG. 6a. The Compromise Standard System of Live Loads for Railway Bridges.

foot. He stated also that, in his opinion, the probable limit of engine would consist of locomotives weighing with tender about two hundred and forty (240) tons on a wheel base of fifty-two (52) feet and having axle loads of thirty-five thousand (35,000) pounds for pilots, seventy thousand (70,000) pounds for drivers, and forty-one thousand (41,000) pounds for tenders. In respect to car loads Mr. Hodge stated that while the heaviest loaded coal cars weigh four thousand (4,000) pounds per lineal foot, there were in use on the Monongahela Connecting Railroad of Pittsburg iron ore cars weighing when loaded seven thousand three hundred (7,300) pounds per lineal foot. Several of the gentlemen who discussed the paper came to the conclusion that a live load involving fifty thousand- (50,000) pound driver axle loads and car loads of five thousand (5,000) pounds per lineal foot is as large as is necessary for railroad bridges.

Such were the opinions of engineers in 1904.

In the summer of 1907, the author, in order to obtain the latest practice concerning railway live loads, sent to the chief engineers of all railroads in the United States, Canada, and Mexico, having a mileage of one thousand miles and upward, the following letter:

> "We are about to make some investigations concerning bridges, the result of which it is our intention to present eventually to the engineering profession; and we find it necessary for our purpose to collect certain data.
>
> "At present we are dealing with the live load question; and we should like to obtain from you a copy of the standard bridge specifications of your railroad, containing the live load diagram that you employ.
>
> "We should be pleased to know whether you deem your standard loading heavy enough for the future as well as for the present, and, if not, to what extent you think it may have to be increased during the next ten years."

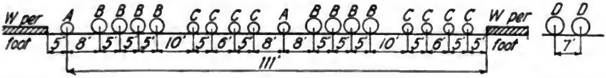
To this letter there were received replies from engineers representing one hundred and fifty thousand (150,000) miles of line; and a compendium and digests of the data accumulated were made, from which it was seen that the average driver axle load then specified by the leading American railroads was fifty-one thousand one hundred (51,100) pounds, and the average car load four thousand nine hundred and sixty six (4,966) pounds per lineal foot. About twenty-five (25) per cent of the railroads that replied to the circular were contemplating an increase in their specified live loads; hence it is evident that the general opinion of the gentlemen who discussed Mr. Hodge's paper concerning the sufficiency of a fifty thousand (50,000)-pound axle load was incorrect, and that Mr. Hodge's suggestion that such a load be considered as a minimum is justified.

Some interesting deductions were made from this compendium concerning the relative values of the various axle loads and the car loading per lineal foot. Calling the driver axle load unity, the average pilot axle load is 0.47, the average tender axle load is 0.65 and the average car load per lineal foot is 0.10.

Based upon the preceding information the live loads tabulated in Fig. 6*b* have been adopted by the author as his new standard.

It will be seen that the loads for the different classes are multiples of each other. This method was adopted because it does not appear to conflict materially with the present average practice of the railroad chief engineers, and because, as before stated, it simplifies the preparation of the curves of equivalent uniform loads and total end shears. The pilot axle load has been made fifty (50) per cent of the driver axle load, the tender axle load has been made seventy (70) per cent of same, and the car load per lineal foot ten (10) per cent thereof. The reason for using a slightly higher per cent than the average for the tender loads is the fact that many railroads are now making a practice of attaching heavy new tenders to their old locomotives.

The new standard has been started with the light loading of Class 40 as an accommodation to the cheap, new railroads of the West that cannot stand the expense of putting in heavy bridges; but the author is advising his clients not to use any lighter load than Class 55, especially for truss bridges, as locomotives of that weight are in constant use on many railroads. The new standard differs from that of Mr. Cooper mainly in the greater tender loads, but also in the lengths of engines and tenders and in the heavier classes adopted for future possibilities. The total length from the first pilot to the cars is only one foot greater than the average shown on the digest table of loadings. This increase is a move in the right direction in view of the fact that the present tendency is to augment the lengths of both locomotives and tenders in order to accommodate their greater weights.



Class	A	B	C	W	D
40	20,000	40,000	28,000	4,000	48,000
45	22,500	45,000	31,500	4,500	54,000
50	25,000	50,000	35,000	5,000	60,000
55	27,500	55,000	38,500	5,500	66,000
60	30,000	60,000	42,000	6,000	72,000
65	32,500	65,000	45,500	6,500	78,000
70	35,000	70,000	49,000	7,000	84,000

FIG. 6*b*. Live Loads for Railway Bridges.

Figs. 6*c*, 6*d*, and 6*e* show the end shears and the equivalent uniform live loads for the new standard. The apparent inconsistency between the readings given in Fig. 6*d* and those in Fig. 6*e* for 100-foot spans is due to the fact that the equivalent uniform loads for plate-girder spans were computed so as to produce the correct moment at mid-span; while in the case of truss spans the equivalent uniform loads were figured so as to give correct moments at the quarter points, calling for a somewhat greater load per foot of span. For further explanation see Chapter X.

In the author's opinion, it is not likely that much heavier loads than Class 65 will be needed at all generally for many years. He is also of the opinion that Class 70 approaches closely the practicable limit for locomotives, tenders, and trains of cars, unless the gauge of railroad be increased; because any materially greater loading would be decidedly top-heavy and, therefore, not only very injurious to the track because of sway but also actually dangerous on account of the tendency to overturn.

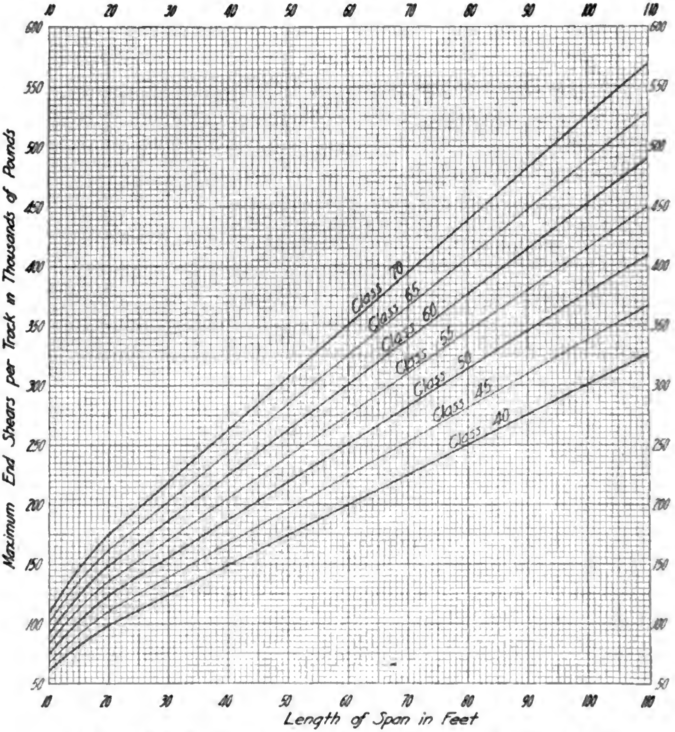


FIG. 6c. Maximum End Shears for Plate-girder Spans of Railway Bridges.

The preceding paragraph was written in 1909; and just six years later an opportunity has occurred for checking its correctness, for in the March, 1915, *Bulletin* of the American Railway Engineering Association there is a timely and interesting paper on "Heavy Locomotive Loadings" by A. C. Irwin, Esq., C. E., of the Engineering Department of the Chicago, Milwaukee, and St. Paul Railway. In it he gives thirteen examples of the heaviest engine loadings per rail and an "equivalent" comparison with an extension of Cooper's "E Loadings," showing that the "Mallet Triplex" loading of the Erie Railway for spans of about 100 feet would just reach Class E72, that the Mallet loading of the Atchison, Topeka, and Santa Fe System (Class 3,000) equals Class E67 for spans ranging from 80 to 100 feet, that the Mallet loading of the Chicago, Burlington, and Quincy Railway (Class M2) is equivalent to Class E66 for spans of 35 to 45 feet, that two other Mallet engine loadings are equivalent to Class E65, that a majority of the loadings equal or exceed Class E60, that all but one exceed Class E55, and that the one exception is equivalent for short spans to Class E50. This shows the necessity for adopting a heavy

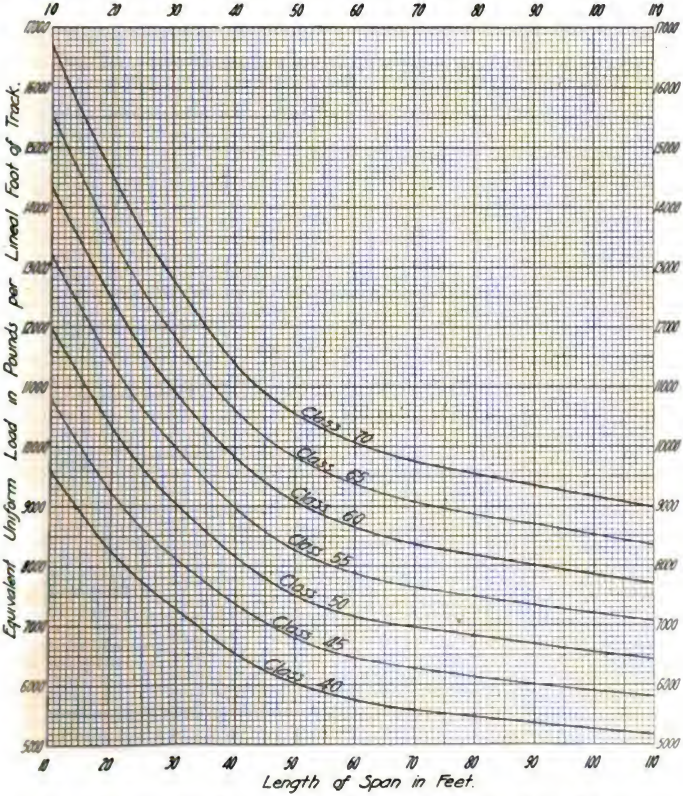


FIG. 6*d*. Equivalent Uniform Live Loads for Plate-girder Spans of Railway Bridges.

live load for the bridges of any line that is likely ever to use engines of the Mallet type.

As the live loads adopted for this treatise are about one per cent heavier than the corresponding live loads of the extended Cooper standard, it is evident that there is today only one live load exceeding the author's

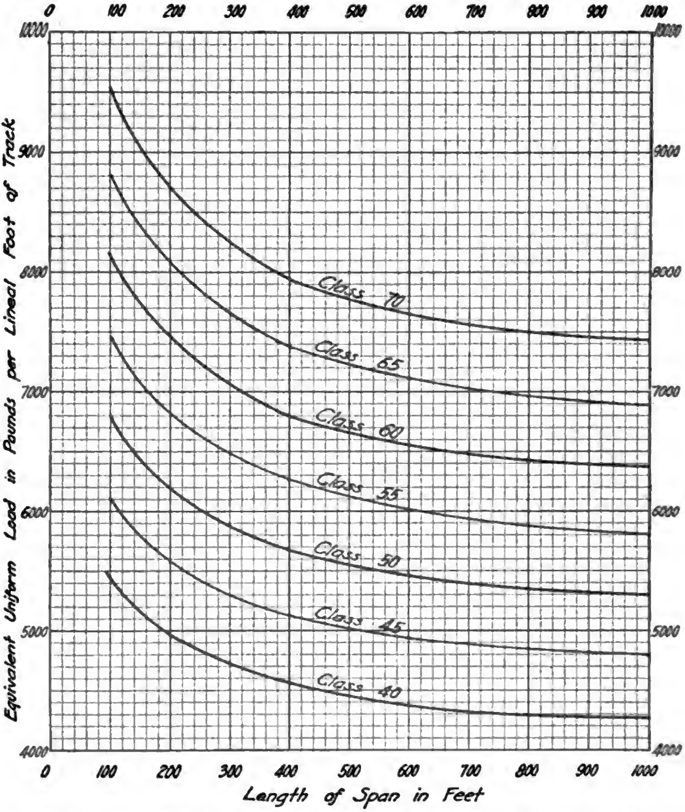


FIG. 6e. Equivalent Uniform Live Loads for Truss Spans of Railway Bridges.

Class 70—and that by merely one per cent and solely for spans of about one hundred feet. It must be remembered that as yet engines of the Mallet type are used on only a few railroads, and not many of them give as great bending moments as does Class 65 of the live loads adopted by the author; hence his prognostication of six years ago has been verified.

In double-track bridges, in accordance with the theory of probabilities, it is permissible and often advisable to use more than a single class of live load, one for stringers, a lighter one for floor-beams, and in long spans a still lighter one for trusses. The reason for this is because, while the stringers are likely to receive occasionally the heaviest engine loading, the floor-beams are not liable to have to support more than once in a great while two such engines with their wheels in the worst possible position; and because, while a train on one track as heavy as the standard live load is a bare possibility, such a train on each track is a condition that will probably never exist, and the longer the span the greater will be the variation between the actual greatest loading and that composed of the two standard live loads.

Not many years ago but little or no attention was paid to live loads for street railways on bridges, as the cars were so short and light that their weight did not exceed that of a very heavy wagon, and was much less than that of a road-roller. In those days the usual way to provide for the carrying of a street railway over a highway bridge was to double the joists beneath the rails. Today the circumstances are quite different, for, since the advent of electric railroads, the street railway live loads have been steadily on the increase; and now there are in use cars about sixty feet long that will weigh when fully loaded one hundred and twenty-five thousand (125,000) pounds, or about two thousand (2,000) pounds per lineal foot. In order to accommodate all types of street railways and interurban roads the author has adopted the standard live loads for bridges carrying electric railways shown in Fig. 6*f*.

CLASS	AXLE SPACING							AXLE LOAD. (In pounds)
15		5	5	20	5	5		15,000
20		6	5.5	23	5.5	6		20,000
25		7	6	26	6	7		25,000
30		8	6.5	29	6.5	8		30,000
35		9	7	32	7	9		35,000
40		10	7.5	35	7.5	10		40,000

FIG. 6*f*. Live Loads for Electric Railway Bridges.

These six classes ought to suffice for many years to come; for, as far as the author knows, there are no city or suburban railway cars in existence which, when fully loaded, will quite come up to Class 35. As for the number of cars per train, it is assumed that it is either limited to two or that it is a matter to be decided for each individual case as it arises. Under no condition should the number be less than two, and it is improbable that it would ever reach as high as six, consequently the diagram of equivalent uniform loads for each class has been figured for two, three, four, five, and six cars in the train. These curves are shown in Figs. 6*g* to 6*n* inclusive.

The lack of agreement between the readings given in Fig. 6*h* and those in Figs. 6*i*, 6*j*, 6*k*, 6*l*, 6*m* and 6*n* for 100-foot spans is accounted for by the difference in the methods of computing equivalent uniform live loads, as explained previously in this chapter in the case of steam railway loadings and also in Chapter X.

Live loads for highway bridges usually consist of so many pounds per square foot of floor with an alternative loading (that seldom affects more than the floor system and the primary truss members) consisting of a road-roller, a traction engine, or a very heavily loaded motor wagon or lorry.

For many years it has been conceded that the greatest live load which can come upon a highway bridge consists of a crowd of people. Such a crowd collected promiscuously is not likely to weigh more than eighty (80) pounds per square foot of floor, but an unusual jam might easily increase it to one hundred (100) pounds, or even more. A number of experiments have been made to ascertain what weight of men could be crowded into a limited area. Old experiments gave loads varying from eighty-four (84) to one hundred and twenty (120) pounds per square foot; but some modern ones show much higher results, reaching in one case to about one hundred and eighty (180) pounds. Considering the fact that for reasons involving personal comfort, people will not permit themselves to be crushed in a crowd, it is evident that in determining live loads for highway bridges there is no necessity for assuming extraordinary conditions which are extremely unlikely to exist. Moreover, it must be remembered that with a densely packed crowd of people there will be little or no impact, owing to the fact that the motion of such a concourse is either very small or practically *nil*. As modern bridge designing involves almost universally the addition to the assumed live loads of a variable allowance for impact, which in the case of very short, narrow highway spans amounts to about sixty (60) per cent, it is evident that the ignoring of excessively heavy crowds of people in proportioning bridges is perfectly legitimate. Again, in bridge designing the theory of probabilities, or, in other words, sound common sense, must be employed when specifying live loads; for it is evident that a long span is not so likely to be loaded to the limit as is a short one, and that the longer the span the smaller should be the live load per square foot of floor adopted. Should, in any extreme and unusual case, the specified live load be exceeded somewhat, no harm would be done, as the result would simply be a short-lived encroachment on what used to be termed the "factor of safety."

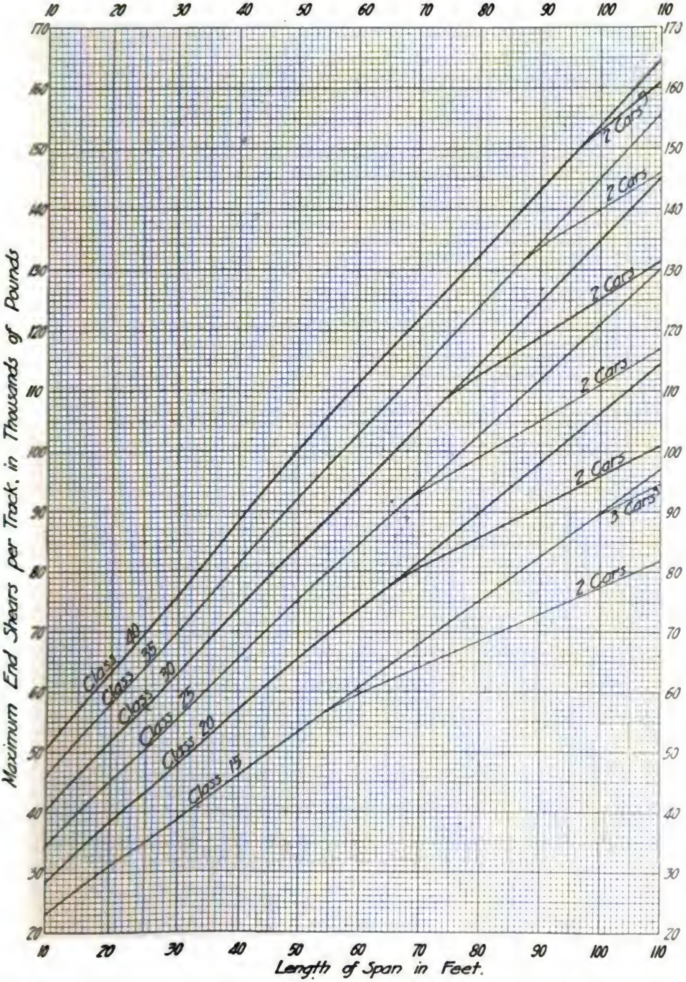


FIG. 69. Maximum End Shears for Plate-girder Spans of Electric Railway Bridges.

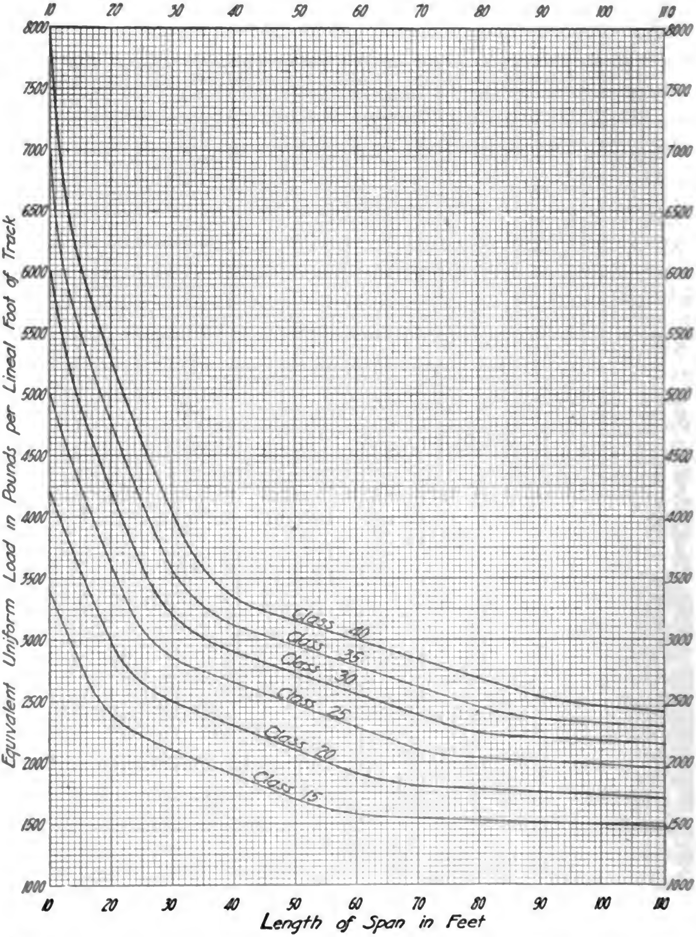


FIG. 6h. Equivalent Uniform Live Loads for Plate-girder Spans of Electric Railway Bridges.

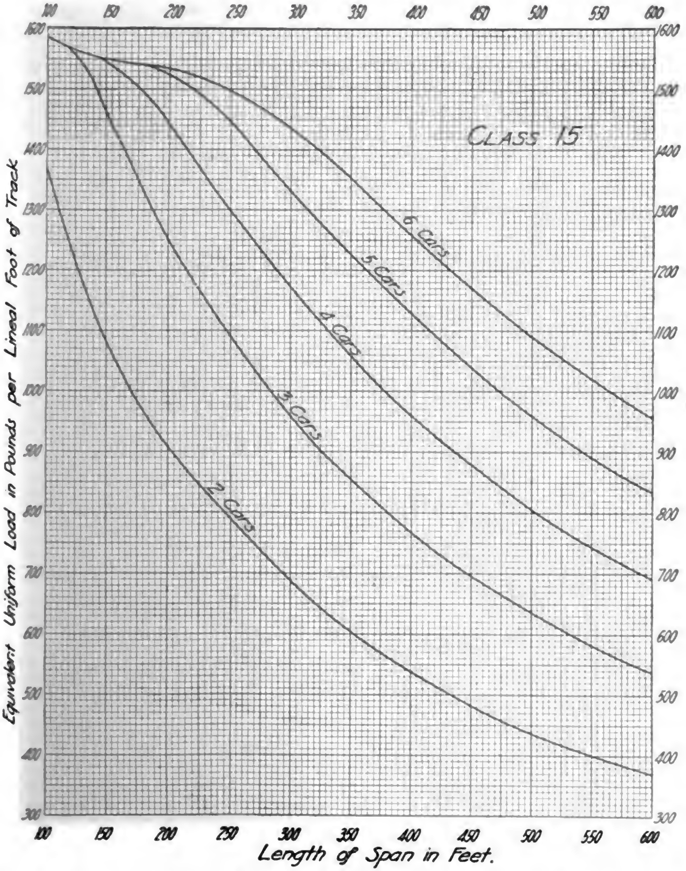


FIG. 6*i*. Equivalent Uniform Live Loads for Truss Spans with Class 15 Electric Railway Loading.

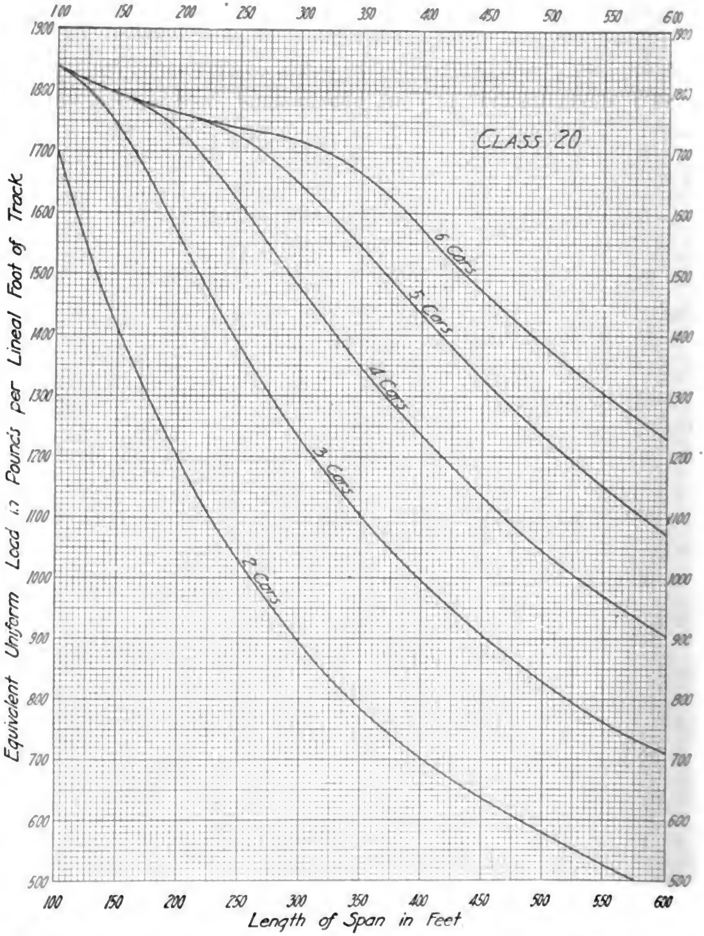


FIG. 6j. Equivalent Uniform Live Loads for Truss Spans with Class 20 Electric Railway Loading.

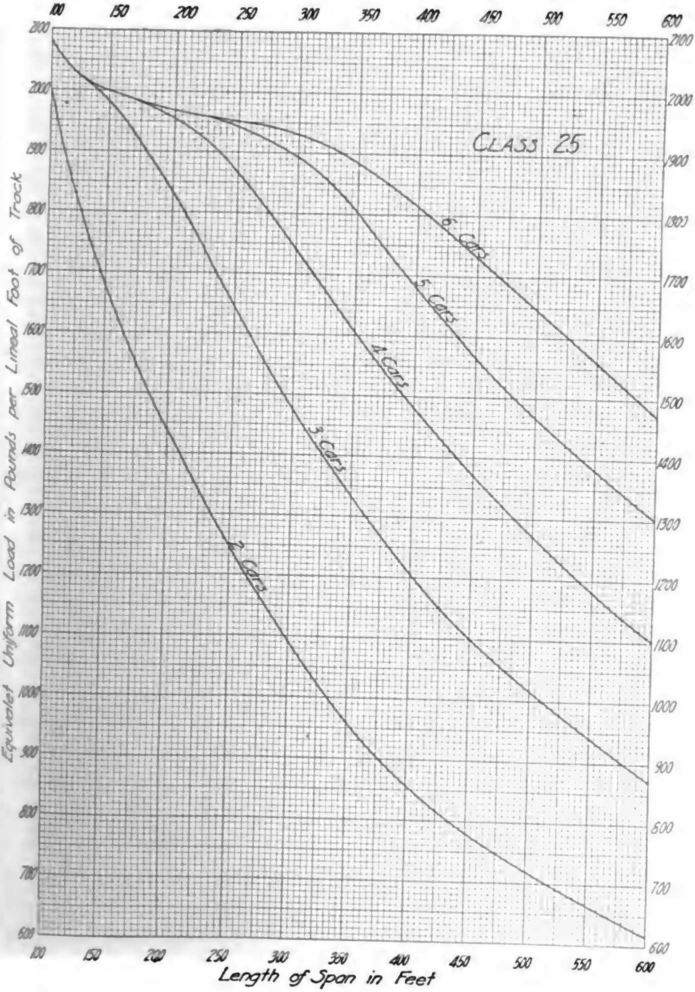


FIG. 6k. **Equivalent Uniform Live Loads for Truss Spans with Class 25 Electric Railway Loading.**

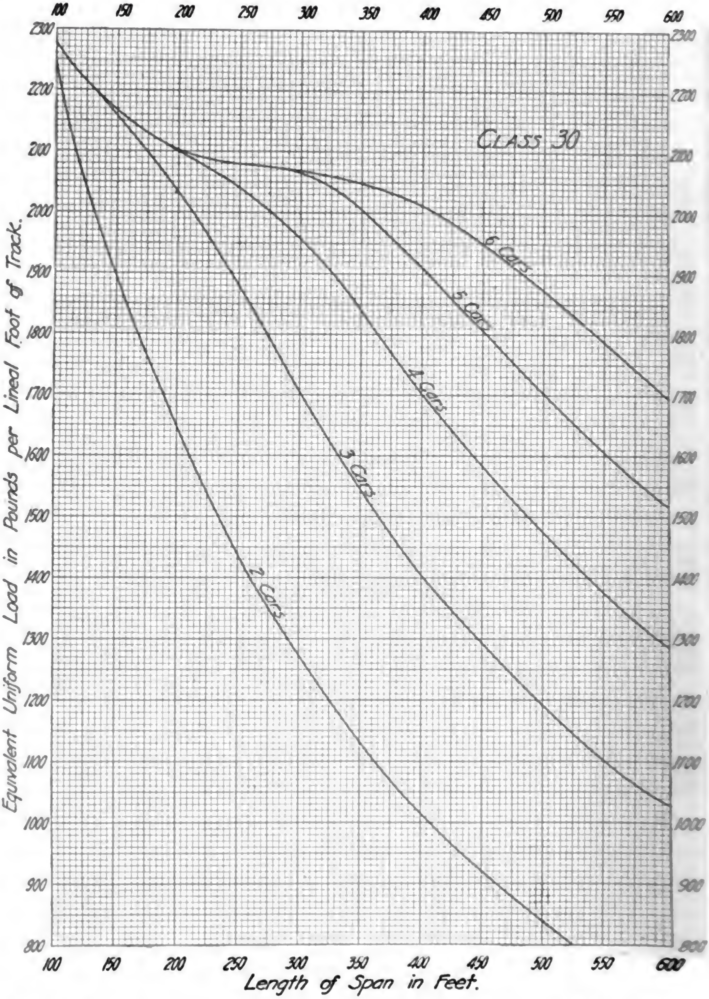


FIG. 61. Equivalent Uniform Live Loads for Truss Spans with Class 30 Electric Railway Loading.

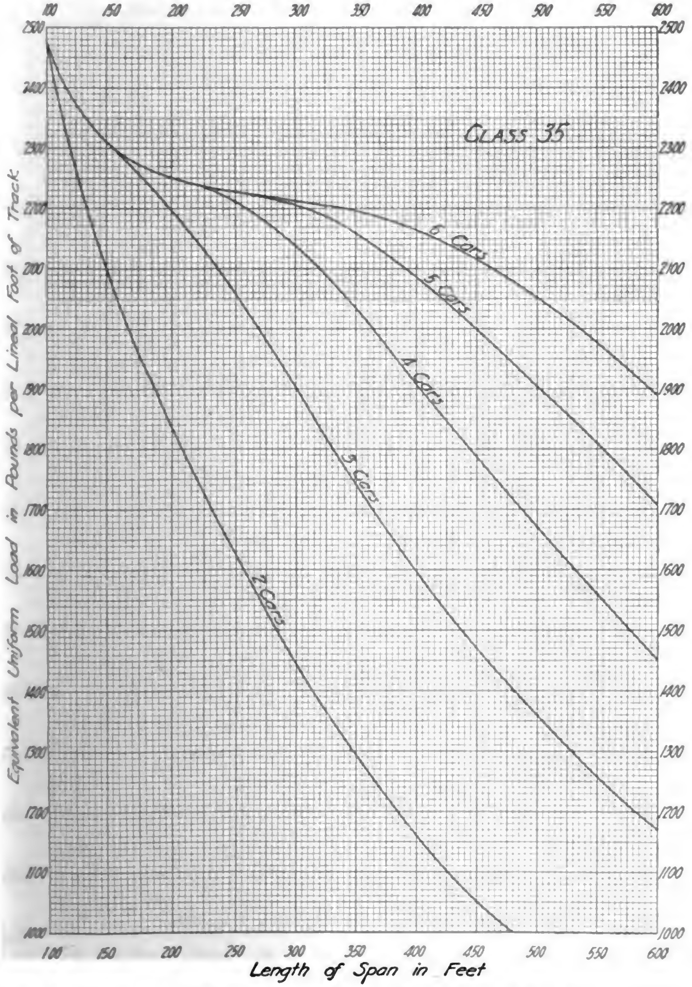


Fig. 6*m*. **Equivalent Uniform Live Loads for Truss Spans with Class 35 Electric Railway Loading.**

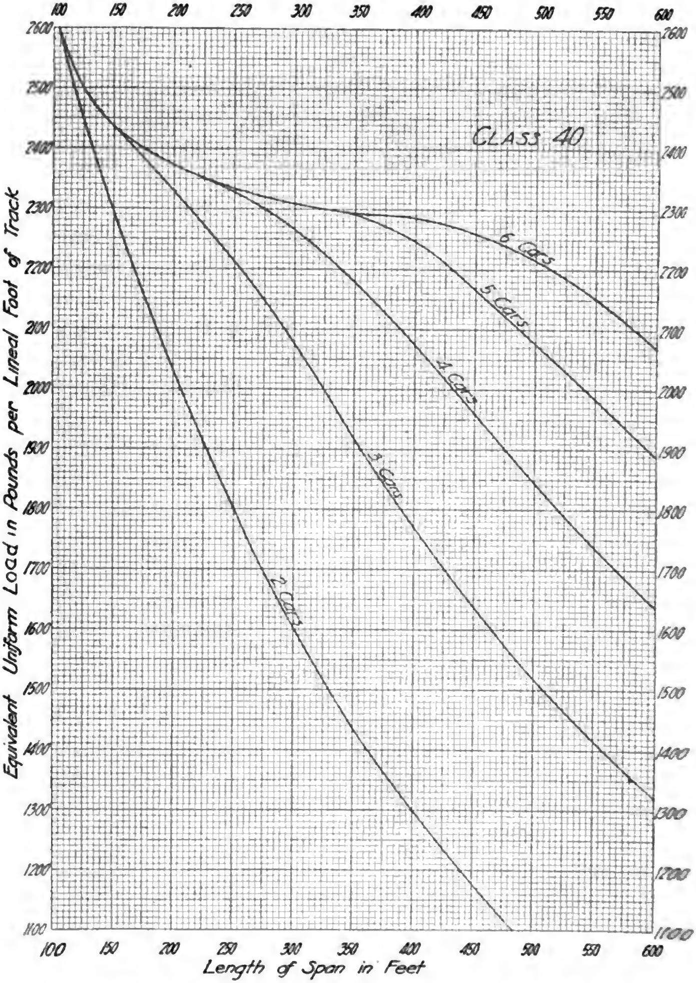


FIG. 6*n*. Equivalent Uniform Live Loads for Truss Spans with Class 40 Electric Railway Loading.

The uniformly distributed, highway, live-loads adopted for the specifications of this treatise, given in Fig. 6*o*, are those of Waddell's *De Pontibus.* They vary for Class A from one hundred and twenty (120) pounds per square foot of floor in very short spans to sixty (60) pounds per same in spans of seven hundred and fifty (750) feet. The corresponding loads for Class B are one hundred (100) pounds and fifty (50) pounds; and those of Class C eighty (80) pounds and forty (40) pounds.

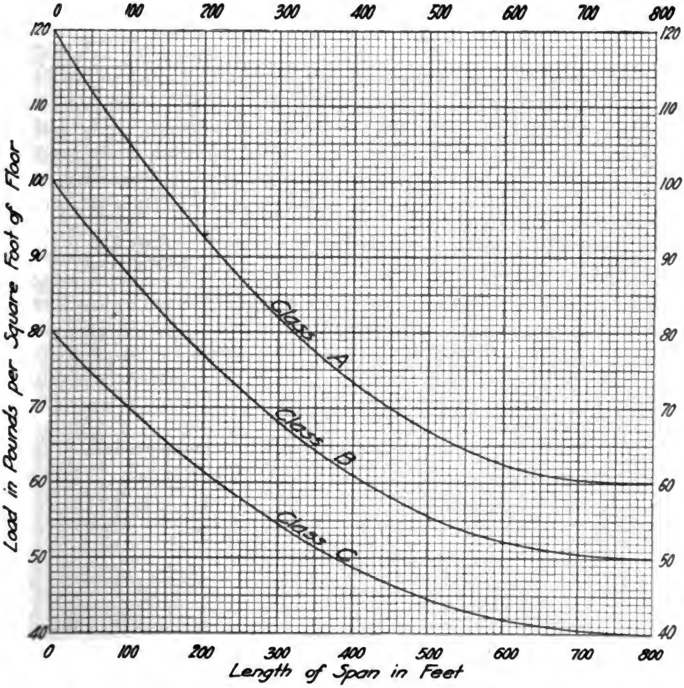
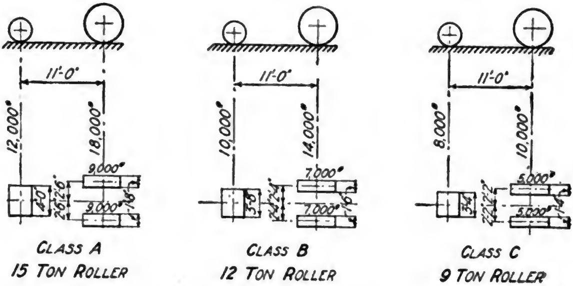


FIG. 6*o*. Uniformly Distributed Live Loads for Highway Bridges.

Some people have an idea that a herd of cattle will weigh more per square foot of space covered than a crowd of people, but such is not the case, as the actual limit for the former is about sixty (60) pounds per square foot. However, the impact from cattle is likely to exceed that from people. The greatest impact comes from soldiers marching in unison, and this is so well known that in crossing bridges they are, by army regulations, made to break step. As soldiers marching in time are never crowded closely, it is evident that their load with its impact can never be injurious to any well-proportioned highway structure, unless it be a suspension bridge in which the rhythm might induce excessive oscillation. This matter of impact will be treated fully in the next chapter.

Until recent years the concentrated live loads for highway bridges consisted only of those from road rollers, traction engines, or heavily

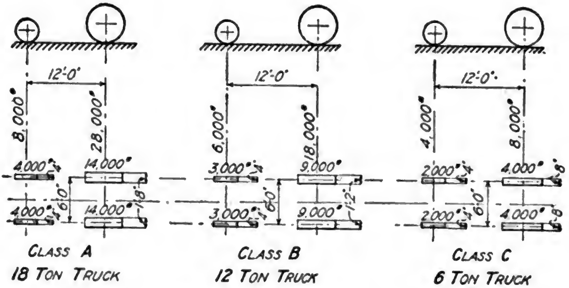


CLASS A
15 TON ROLLER

CLASS B
12 TON ROLLER

CLASS C
9 TON ROLLER

ROAD ROLLER LOADINGS



CLASS A
18 TON TRUCK

CLASS B
12 TON TRUCK

CLASS C
6 TON TRUCK

MOTOR TRUCK LOADINGS

FIG. 6p. Road Roller and Motor Truck Loadings for Highway Bridges

loaded wagons, but today the most important of all is that from motor-trucks or lorries. Almost all of the old highway bridges are incapable of carrying these new live loads with safety. In *Engineering News* of September 3, 1914, there is a paper by Messrs. Manville and Gastmeyer in which is given much valuable information on the subject. In it attention is called to the fact that, even if truck-loadings are apparently not as great as those caused by road-rollers, their effects are more destructive because, on account of their high passing speed, they involve much more impact than the road-rollers do with their slow motion. These gentlemen assume an impact of twenty-five per cent for road-rollers and from forty to fifty per cent for trucks. The author is of the opinion that the road-roller cannot produce any appreciable impact because of its extremely slow speed, but it might occasion a slight jar by rolling over an obstacle too large to be crushed or driven into the floor. As explained fully in the following chapter and in the specifications of Chapter LXXVIII, there is no impact allowance at all for road roller loadings. The impact assumed for trucks by Messrs. Manville and Gastmeyer checks very closely with the formula for highway bridge impact adopted in this treatise.

Fig. 6*p* shows the three classes of road-roller and truck loadings adopted as the author's standard.

In Chapter LXXVIII are given specifications as to how live loads on sidewalks of highway bridges are to be treated. The rules there laid down are based upon the theory of probabilities; for while one sidewalk of a short span (or a panel or two thereof in a long span) might be fully loaded simultaneously with the main roadway while the opposite sidewalk is empty, such a combination of circumstances is not at all likely to occur for any great length of structure.

The subject of combining for any one bridge live loads caused by the various kinds of loading is treated in Chapter XIII.

CHAPTER VII

IMPACT LOADS

To the eminent American bridge engineer, C. C. Schneider, Esq., Past President of the American Society of Civil Engineers, is due the method of proportioning bridge members for the impact produced upon them by rapidly moving loads. It was in 1887 that he wrote his bridge specification for the Pencoyd Iron Company, in which he increased the live load by using an impact formula, and then allowed the same unit stresses for both live and dead loads. This is the only truly scientific and correct method of designing bridges. It is more accurate than any other, the degree of accuracy attainable, of course, being dependent upon the amount of study given to the action of spans and their members under live loads passing at various velocities.

Mr. Schneider's formula was,

$$I = \frac{300}{L + 300},$$

where I is the coefficient for impact and L is the length in feet of the portion of the span covered by the moving load when the member under consideration receives its greatest live-load stress. If we make $L = 0$ in the equation, I will equal unity. This is in accordance with the well-known principle that a load suddenly applied from rest produces twice the effect of the same load applied statically. This impact formula of Mr. Schneider's is widely used even today when, from numerous experiments, it is known to give results too small for short spans and too great for long ones.

The history of the evolution of the determination of impact is as follows:

In the résumé of the discussions of his paper on "Some Disputed Points in Railway Bridge Designing," written in 1891 and published in the *Transactions* of the American Society of Civil Engineers for 1892, the author wrote thus under the heading "Intensities of Working Stresses":

> "In respect to this subject it appears that we are all at sea; and we are liable to remain there until such time as the much needed experiments on actual intensities of working stresses, that I have been advocating for years, be made, after which we shall be able to settle upon a system of intensities that will be logical. Meanwhile we shall have to jog along in the best way that we can, letting each engineer use his own judgment concerning the intensities to employ, or perhaps (which is the best thing to be done under the circumstances) obtain a consensus of opinion as to what system to adopt for a

temporary expedient. The method adopted by Mr. Schneider, of reducing all live-load stresses to their equivalent static stresses before applying a constant intensity, is undoubtedly the scientific way to proportion bridges; but until we have some real knowledge of the effects of dynamically applied loads, it does not seem advisable to develop a system that in all probability will have to be considerably modified in the future.

"It would not be such an immense undertaking to make an exhaustive series of tests of the effects of live loads applied to bridge members with varying velocities. Perhaps a year's time and an expenditure of, say, $50,000 would suffice. If not, more time and money should not be begrudged upon such an important matter. The United States Government is willing to appropriate annually millions of dollars for the United States Engineers to use in experiments upon hydraulic problems. Why cannot the bridge engineers and the railroads obtain from Congress a small appropriation to decide one of the most vital questions in bridge building? If the United States Government refuse to make such an appropriation, cannot one of America's millionaires be persuaded to donate the money as a contribution to applied science?

"In my opinion the proper steps to take after obtaining the money would be as follows:

"First.—To appoint a committee of seven members of the American Society of Civil Engineers, who are acknowledged bridge experts, to act as an advisory board, and let them lay out the series of tests (to be modified later if they should think advisable), appoint a committee of three well-paid expert bridge engineers to make the tests under their instructions, attend to all payments of money, make arrangements with railroad companies for the use of their lines and bridges in making the tests, etc.

"The first practical step to take would be to investigate all the machines thus far invented for measuring extensions and compressions in bridge members, so as to decide upon what kind of apparatus to adopt, or to design new ones if necessary. These machines should be tested thoroughly to determine their accuracy as far as static loads are concerned and to prove their reliability in case of dynamically applied loads. After the machines are shown to be satisfactory, experiments should be begun systematically upon all parts of bridges of modern design, with trains varying in velocity from zero to the greatest attainable speed. Sufficient tests of all kinds should be made to give good average results. Both tension and compression members should be experimented upon, and if the machines prove to be very accurate, even such intricate problems as the distribution of stress in plate girders might be solved. This field of experiment is most inviting, especially because of the great utility of the results; hence there would be no difficulty in obtaining an expert committee to make the tests. Mr. Wolfel's remarks on the subject of measuring the actual intensities of working stresses are most interesting, and are worthy of a careful perusal."

Again, in 1896, when preparing the introductory chapter of *De Pontibus*, the author wrote thus:

"The uncertainty as to the magnitude of the effect of impact on bridges has for many years been a stumbling-block in the path of systemization of bridge designing, and will continue to be so until some one makes an exhaustive series of experiments upon the actual intensities of working stresses on all main members of modern bridges of the various types. The making of these experiments has long been a dream of the author's, and it now looks as if it would amount to more than a mere dream; for the reason that the general manager of one of the principal Western railroads has agreed to join the author in the making of a number of such experiments on certain bridges of the author's designing, the railroad company to furnish the train and all facilities, and the general manager and the author to provide the apparatus and experimenters. It is only lack of time that has prevented these experiments from being made this year, and it is expected that they will be finished in 1898. It is hoped that the result of the experiments will be either to determine a proper formula or curve of percentages of impact for railroad bridges, or else to inaugurate a series of further experiments that will determine it.

"Meanwhile the author has adopted temporarily the formula given in Chapter XIV., viz.,

$$I = \frac{40,000}{L + 500},$$

in which I is the percentage for impact to be added to the live load, and L is the length in feet of span or portion of span that is covered by the said load.

"This formula was established to suit the average practice of half a dozen of the leading bridge engineers of the United States, as given in their standard specifications, and not because the author considers that it will give truly correct percentages for impact.

"In spite of all that has been said to the contrary in the past or that may be said in the future, the impact method of proportioning bridges is the only rational and scientifically practical method of designing, even if the amounts of impact assumed be not absolutely correct, for the said method carries the effect of impact into every detail and group of rivets, instead of merely affecting the sections of the main members, as do the other methods in common use.

"The assumption made in some specifications that the live load is always twice as important and destructive as the dead load, irrespective of whether the member considered be a panel suspender or a bottom chord-bar in a five-hundred-foot span, is absurd, and involves far greater errors than those that would be caused by any incorrectness in the assumed impact formula.

"The author acknowledges that he anticipates finding the values given by the formula somewhat high; but it must be remembered that the said formula is intended to cover in a general way, also, the effects of small variations from correctness in shop work, or to provide for what the noted bridge engineer, the late C. Shaler Smith, used to term the factor of ignorance."

The series of experiments mentioned in this last quotation, unfortunately, failed to materialize, owing to the death of the railroad manager who was to join in the investigation.

A few years later the author, when examining a large number of bridges on the International and Great Northern Railway of Texas, took advantage of the opportunity to measure the deflection of numerous spans under live loads at different velocities. He had prepared for the purpose a home-made deflectometer of rather crude design and operated by hand, but quite satisfactory in respect to recording. Unfortunately, he was not possessed of an extensometer. As long ago as 1885, when in Japan, he had designed and manufactured one in the shops of the Imperial University at Tokyo; but it did not record satisfactorily, owing, undoubtedly, to defects in design as well as to crudeness in workmanship.

The International and Great Northern Railway spans tested varied in length from about fifty feet to two hundred feet, and the impact percentages on the spans as a whole were, in general, from fifty to twenty per cent. The author had at his disposal a train consisting of a large locomotive, a heavily loaded freight car, and a caboose. From this series of tests he learned that, except in the case of comparatively short girders, for any particular span and any particular loading there is a velocity of train, less than the greatest attainable, which gives maximum impact effect. He learned also a very curious fact, viz., that for identical superstructures the amount of impact is dependent upon the character of the supporting piers. This he discovered accidentally thus: There were at various places on the line a large number of 159-ft., pin-connected spans built exactly alike, and he had tested several of them by deflection, finding in each case almost exactly twenty per cent for the maximum impact. When he was just about to complete his field work by inspecting a long bridge containing several spans of different lengths supported on cylinder piers, he found that he had some spare time while waiting for a passing train, and as there were two of the said 159-ft. spans in the structure, he occupied himself by measuring the deflection of one of them. Much to his surprise he found an impact of forty per cent, and thinking that there must be some mistake in his record, he tried again and discovered the same result. Not content with the showing thus obtained, he moved the apparatus to the other identical span and found forty per cent once more. Suddenly it dawned upon him that all the similar spans tested previously had been supported upon stone-masonry substructure; and from that fact he concluded that the greater impact he was finding must be caused by the flimsiness and the vibratory character of the small cylinder piers. This confirmed him in an opinion which he had long held and expressed, viz., that ordinary cylinder piers are unfit for the substructure of railroad bridges.

No action of any importance was taken in America to study systematically the question of impact on railroad bridges until 1907, when the American Railway Engineering Association appointed a committee to make a thorough investigation of it, although some desultory experiments like those of the author had been made from time to time by a few American and European engineers. The committee referred to issued its report in 1911, indicating that it had experimented upon 21 plate-girder spans up to 100 ft. in length and upon 24 truss spans from 100 to 250 ft. in length, using generally enough loaded cars to cover the span tested, and employing speeds from ten to sixty miles or more per hour. From that report are taken the following statements and conclusions, but as the author has condensed them, they will not usually be printed with quotation marks:

For speeds under ten (10) and even fifteen (15) miles per hour the recorded impact was practically zero. This information should have a large money value for railroad companies; because it has hitherto been the custom, when placing a slow order upon a bridge of doubtful carrying capacity, to limit the speed to four or five miles per hour. By changing this limit to twelve or fifteen (or say ten, so as to provide for possible breaches of the rule) much time could be saved for all trains—and in railroad operation, as in most other kinds of business, "time is money."

The chief factors in causing impact are unbalanced locomotive drivers, rough and uneven track, flat or irregular wheels, eccentric wheels, rapidity of application of load, and deflection of beams and stringers. The term impact very properly was considered by the committee to include any effect of the moving load which results in stresses exceeding the static stresses. With the track and the rolling stock in good shape, the chief cause of impact is the unbalanced condition of the drivers of the ordinary locomotive. This condition does not exist in the balanced four-cylinder locomotives or in electric locomotives.

The committee corroborated the fact discovered long ago by the late Prof. S. W. Robinson that the maximum impact on a bridge is dependent upon how its normal rate of vibration coincides with the times of the series of impulses from the unbalanced driver loads. Such cumulative effect of impulses cannot occur for bridges of very short span-length, because the normal rate of vibration of such structures is higher than the rate of rotation of the drivers at the highest practicable speeds. It does occur in spans as short as seventy-five feet. The speed at which the impulses referred to show a cumulative effect are termed the critical speed; and it is this speed which produces the greatest impact on the span. The time of vibration of any span is given by the formula,

$$T = \sqrt{\frac{(W+P)\,d}{P}},$$

in which T = time of vibration of loaded structure in seconds;
W = dead load per foot, assumed as uniform;
P = live load per foot, assumed as uniform;
d = static deflection in feet due to load P, as determined by direct measurement.

The critical speeds observed agreed very well with those calculated by the formula, varying from 65 miles per hour for 60-ft. spans to 25 miles per hour for a 300-ft. span and to 20 miles per hour for a 440-ft. span. They corroborated the results of the author's experiments on the International and Great Northern Railway bridges, in which he found for spans of 150 or 160 ft. critical speeds of 35 or 40 miles per hour.

The committee noted, as the author anticipated many years ago, that the impact on the main members of a span is greater than that upon the structure as a whole, or, in other words, the extensometer measurements gave somewhat larger results than the deflectometer measurements. Fig. 7a records the greatest percentages of impact found for the various spans tested by the committee. It gives also two suggested impact curves and the formulæ from which they were computed, viz.,

$$I = \frac{100}{1 + \frac{L^2}{20{,}000}} \quad \text{and} \quad I = \frac{60}{L}.$$

The author has taken the liberty of recording upon the same diagram a curve of impact for single track steam railway bridges, computed by a formula which he has evolved, viz.,

$$I = \frac{165}{L + 150}.$$

This formula is discussed further on in this chapter.

The committee noted that a wide spacing of stringers caused the ties to afford a cushioning effect which reduced materially the impact as compared with that given by a solid steel floor or by a floor in which the rails are carried either directly over the stringers or are attached without any intermediary to closely spaced steel cross-girders. They found also that the effect of a ballasted floor in reducing shock was even more marked than that of wide stringer spacing.

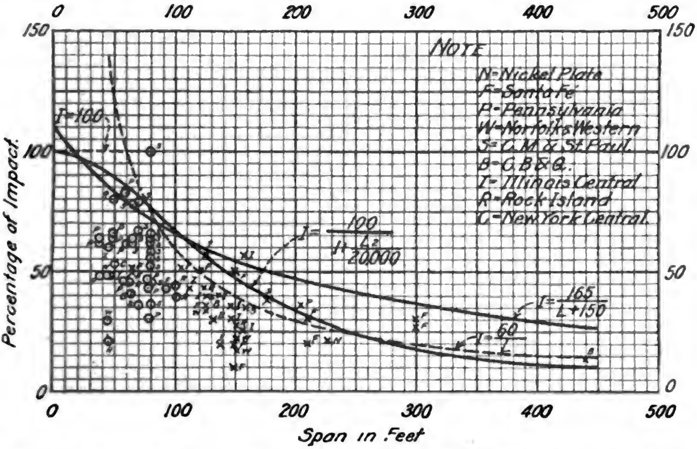


FIG. 7a. Maximum Impact Percentages by Actual Tests on Plate-girders and Main Members of Truss Spans upon Various Railroads.

The committee summarized the results of its work as follows:

> "(1.) With track in good condition the chief cause of impact was found to be the unbalanced drivers of the locomotive. Such inequalities of track as existed on the structures tested were of little influence on impact on girder flanges and main truss members of spans exceeding 60 to 75 feet in length.
>
> "(2.) When the rate of rotation of the locomotive drivers corresponds to the rate of vibration of the loaded structure, cumulative vibration is caused, which is the principal factor in producing impact in long spans. The speed of the train which produces this cumulative vibration is called the 'critical speed.' A speed in excess of the critical speed, as well as a speed below the critical speed, will cause vibrations of less amplitude than those caused at or near the critical speed.

"(3.) The longer the span-length the slower is the critical speed; and, therefore, the maximum impact on long spans will occur at slower speeds than on short spans.

"(4.) For short spans, such that the critical speed is not reached by the moving train, the impact percentage tends to be constant so far as the effect of the counterbalance is concerned, but the effect of rough track and wheels becomes of greater importance for such spans.

"(5.) The impact as determined by extensometer measurements on flanges and chord members of trusses is somewhat greater than the percentages determined from measurements of deflection, but both values follow the same general law.

"(6.) The maximum impact on web members (excepting hip verticals) occurs under the same conditions which cause maximum impact on chord members, and the percentages of impact for the two classes of members are practically the same.

"(7.) The impact on stringers is about the same as on plate-girder spans of the same length, and the impact on floor-beams and hip verticals is about the same as on plate girders of a span length equal to two panels.

"(8.) The maximum impact percentage as determined by these tests is closely given by the formula,

$$I = \frac{100}{1 + \frac{l^2}{20,000}},$$

in which I = impact percentage and l = span-length in feet.

"(9.) The effect of differences of design was most noticeable with respect to differences in the bridge floors. An elastic floor, such as furnished by long ties supported on widely spaced stringers, or a ballasted floor, gave smoother curves than were obtained with more rigid floors. The results clearly indicated a cushioning effect with respect to impact due to open joints, rough wheels, and similar causes. This cushioning effect was noticed on stringers, floor beams, hip verticals, and short-span girders.

"(10.) The effect of design upon impact percentage for main truss members was not sufficiently marked to enable conclusions to be drawn. The impact percentage here considered refers to variations in the axial stresses in the members, and does not relate to vibrations of members themselves.

"(11.) The impact due to the rapid application of a load, assuming smooth track and balanced loads, is found to be, from both theoretical and experimental grounds, of no practical importance.

"(12.) The impact caused by balanced compound and electric locomotives was very small, and the vibrations caused under the loads were not cumulative.

"(13.) The effect of rough and flat wheels was distinctly noticeable on floor-beams, but not on truss members. Large impact was, however, caused in several cases by heavily loaded freight-cars moving at high speeds."

In *Engineering News* of August 1, 1912, there is a paper entitled "A New Impact Formula," by Gustav Lindenthal, Esq., C. E., which contains much valuable information; but the formula proposed is far too complicated, being based on many theoretical assumptions. Moreover, some of the statements and deductions which it contains are not in accord with the latest experiments on impact—those made by the committee of the American Railway Engineering Association as herein previously described. For instance, there are no experiments on record to show that for a rail supported on ties "the effect of live load and impact is equivalent to three times the effect of the quiescent load." This paper ignores the first principle that "Simplicity is one of the highest attributes of good designing."

There is a paper in the 1912 *Transactions* of the American Society of Civil Engineers entitled "Specifications for the Design of Bridges and Subways," by Henry B. Seaman, Esq., C. E., which contains some material concerning impact that is worthy of perusal, especially the discussion by Victor H. Cochrane, Esq., C. E. Fig. 7*b* contains seven impact curves collected and diagrammed by Mr. Cochrane; and the author has again taken the liberty of plotting here the curve derived from his proposed formula for single-track railway bridges, viz.,

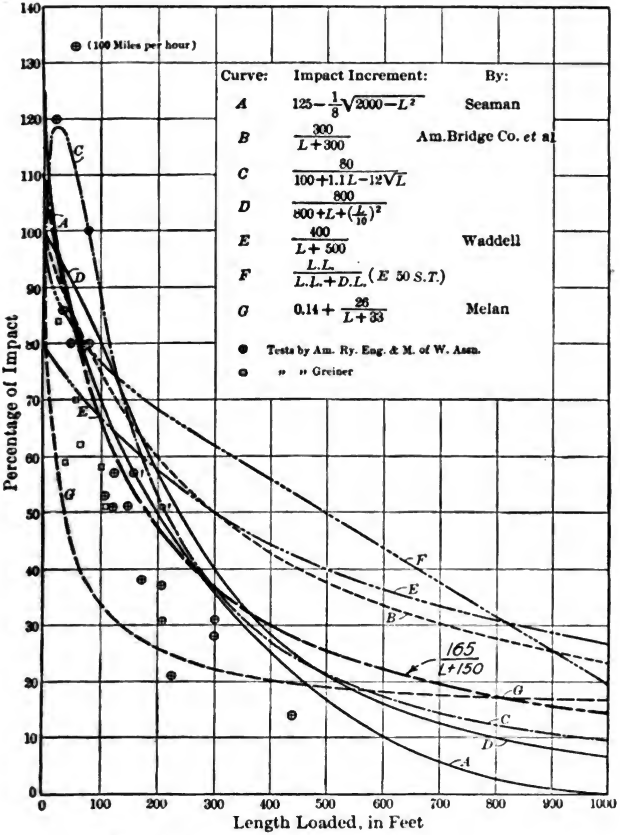


FIG. 7*b*. Impact Tests and Various Impact Curves.

$$I = \frac{165}{L + 150}.$$

It is submitted that this curve is preferable to any of the others; for while it keeps just above the plotted points for short spans, it does not approach so near the zero line for long spans as do the three other curves which follow the test points more or less closely. The author is firmly of the opinion that the effect of impact never is zero, no matter how long the span, when the speed of the train is unrestricted.

All the experiments on impact thus far, as well as can be learned, have been made upon single-track bridges; and it is evident that for bridges with a greater number of tracks the impact would be less than on those for single track, the larger the number the smaller the impact. While he has had no proper data from which to adjust this variation, the author has evolved the following formula for steam railway bridges having any number of tracks:

$$I = \frac{165}{nL + 150},$$

where I is the coefficient of impact, n the number of tracks, and L the loaded length in feet. Fig. 7*c* gives the curves by this formula for spans from zero to one thousand feet for structures having one, two, three, and four tracks. These curves look reasonable and logical; and if they err at all, the error is on the side of safety. For instance, taking a span-length of one hundred feet, while a single-track structure has an impact of 66 per cent, a double-track structure has one of 47 per cent, a three-track structure one of 36 per cent, and a four-track structure one of 30 per cent. When one considers that in multiple track structures the trains run in opposite directions and that the cumulative vibrations of one train undoubtedly have a tendency to check those of another train, he must conclude that these figures are more than safe.

For electric railway bridges the author after several trials has finally adopted the impact formula,

$$I = \frac{120}{nL + 175}.$$

It is based entirely on engineering judgment, with the impact formula for steam railway bridges as a guide. It, too, undoubtedly errs upon the side of safety. Fig. 7*d* shows for spans varying in length from zero to one thousand feet and for structures having one, two, three, and four tracks the curves derived from this last formula.

For highway bridges the formula finally adopted is

$$I = \frac{100}{nL + 200},$$

where n is the total width of roadway and sidewalks divided by 20—for instance, if the total clear width of deck is 60 feet n will be equal to three (3). Fig. 7*e*, as in the previous cases, gives the corresponding curves for

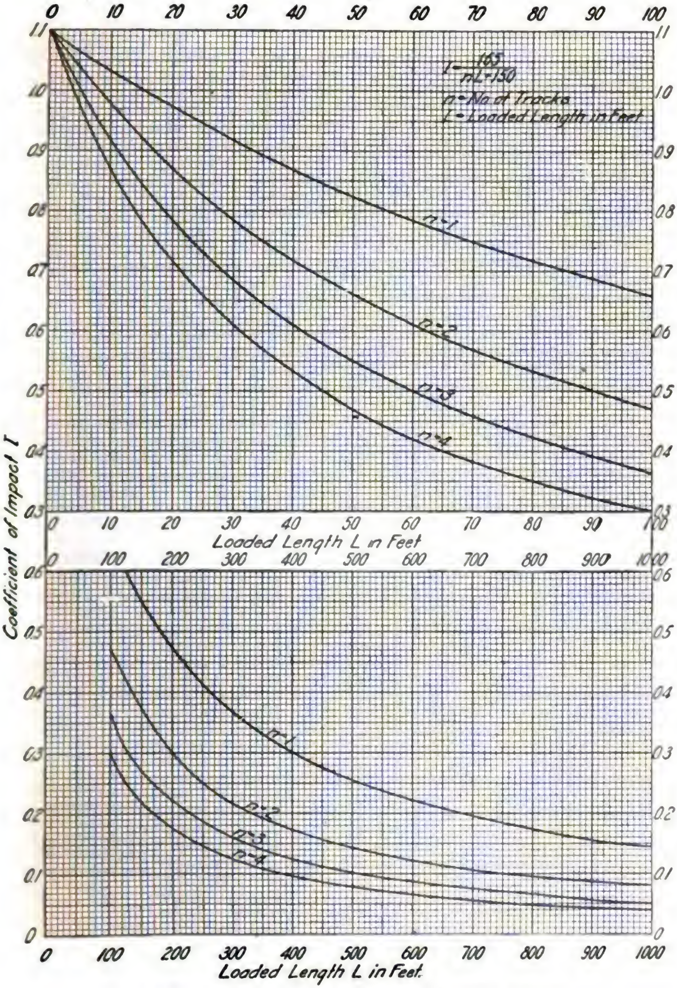


Fig. 7c. Coefficients of Impact for Railway Bridges.

four different values of *n*. If it is fractional, the impact can either be interpolated or be taken from the curve of nearest value.

There is still another impact loading to be considered, viz., that from the dead loads of moving spans. In swings and bascules when the movable span is set in motion or brought to rest suddenly there is a jar or shock which augments the dead-load stresses; and to allow for this effect the said stresses are increased twenty-five (25) per cent in the specifications of Chapter LXXVIII. This increase of dead load does not combine with the live load; nevertheless there are or may be certain main members and details the sections of which it will augment. This dead-load impact when applied to vertical lift bridges will never change the

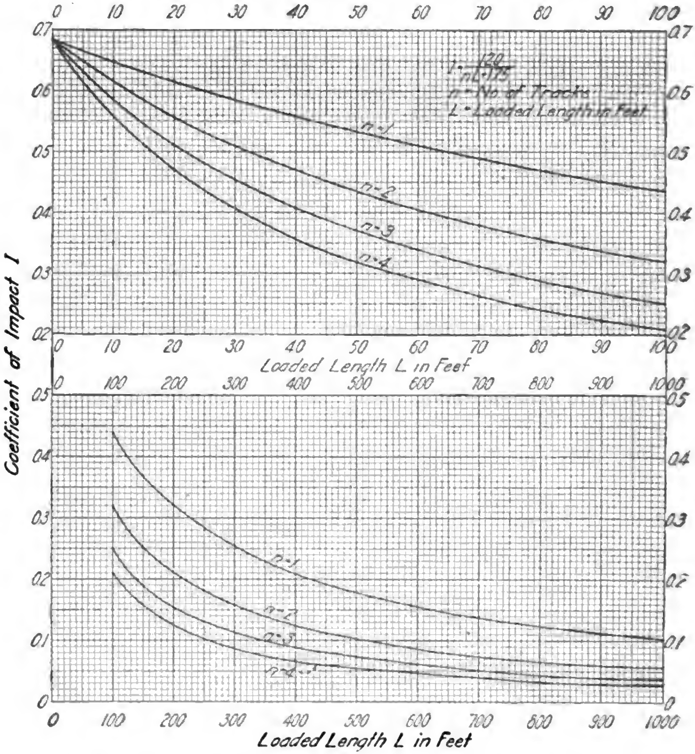


FIG. 7*d*. Coefficients of Impact for Electric Railway Bridges.

sections of the members of the movable span, but it will increase those of the tower columns, the supporting ropes, the equalizers, the hangers, and all the connecting details for these parts.

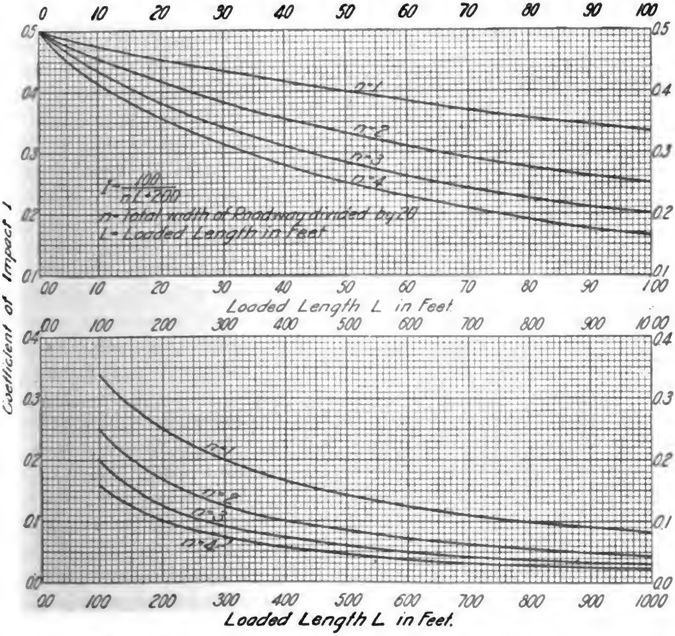


FIG. 7e. Coefficients of Impact for Highway Bridges.

CHAPTER VIII

CENTRIFUGAL FORCE AND OTHER EFFECTS OF TRACK CURVATURE

WHENEVER it is necessary to design a railway bridge on a curve, due account must be taken of the effects upon the structure produced by the curvature of the track. Where a bridge is located on a tangent, its axis is made to coincide with the centre line of track or tracks, and the structure is designed symmetrically about the vertical plane through the said centre line. This gives the same loads on the symmetrically corresponding parts of the span and causes their sections to be alike. With the structure on a curve it is not always possible so to arrange the layout that corresponding members shall have the same loads and sections, although it is usually practicable to make the differences very slight, and in many cases negligible. Even when the differences are considerable, it may prove economical from the standpoint of the shop work to use for the more lightly loaded member the same section as for the one more heavily loaded.

There are four elements that enter into the computations for a bridge on a curve due to the curvature of the track, viz.: the curvature of the track itself, the eccentricity of the track, the centrifugal force, and the superelevation of the outer rail. It is not always obligatory to consider all of these effects, as the necessity for so doing will depend on the location of the structure and the class of traffic passing over it. The effects of the curvature and eccentricity of the track will always have to be taken into account; but those due to centrifugal force and superelevation of outer rail will have to be considered only where the speed of the train demands it. The centrifugal force is directly proportional to the square of the velocity of the train, as is also the superelevation required. The latter is employed in order to overcome the bad effect of the centrifugal force which exists on a curve without superelevation. For low speeds the centrifugal force and the required superelevation are small; and where it is practicable to figure on very low speeds they may be neglected altogether.

The velocity of the train to be assumed in determining the centrifugal force and superelevation will depend on various factors. In the first place the location of the bridge should be taken into consideration. There are various circumstances in connection with the position of a structure that might call for a slow speed. When a bridge is located in a city, it is not uncommon to find speed restrictions. Again, when a bridge is near a crossing, station, or water tank, the speed at which the train crosses the structure will usually be slow. Also when a road crosses a navigable stream on a low bridge, all trains are required by law to come to a full stop before reaching the draw span. If there is reversal of curvature on a bridge, it is very unlikely that a train will cross it at a high speed, especially where the curves are sharp and are connected either by a very short tangent or by no tangent at all. Under such circumstances it is generally necessary to superelevate the outer rail only a slight amount, if any; and there will then be no need of taking into account the effect of superelevation and centrifugal force.

Under other circumstances, however, the speed of the train will depend on the classes of traffic, whether passenger or freight or both, that cross the structure, and upon whether it is on a main or a branch line. The speed will also depend somewhat on the degree of curvature, as it is generally the rule that a train will slacken its velocity as it approaches a curve; and the sharper the curve the greater the retardation.

If every train were to take a given curve with the same maximum speed, the centrifugal force should be figured for that velocity, and the superelevation should be such as to make the resultant of the vertical and the centrifugal loads normal to the plane of the track. This would give equal loads on the two rails—the best possible condition for both the traffic and the structure itself. Such an arrangement is possible where one class of traffic alone uses the line, with the same speeds approximately in both directions, or where separate tracks are provided for each of the two classes of traffic. The latter condition is to be found only on certain of the main lines of the large eastern roads where four and six tracks are employed. As a general rule, either a single or a double track is used, and both classes of traffic pass over the same tracks.

With the two classes of traffic occupying the same rails, it is impossible to adopt a speed that will fit both. The freight trains when loaded always run at a comparatively low velocity, producing a small centrifugal force and requiring a low superelevation, if any. When empty, however, they travel at a much higher speed, requiring a greater superelevation. The passenger trains often run at the highest attainable speeds, requiring much superelevation and producing great centrifugal force. The maximum centrifugal force should be figured for the greatest velocity, properly reduced for the degree of curve. The speed for which the superelevation is figured, however, should be adjusted to the best possible advantage so as to suit both the freight and the passenger traffic. As a rule, the passenger traffic should be given the preference in this, for it is the traffic that receives the greatest inconvenience from unbalanced centrifugal force. As stated before, if passenger traffic practically monopolizes the line, a superelevation should be provided for approximately the maximum velocity. But where heavy freight is also handled over the road, such a superelevation would be excessive for this class of traffic, forcing the wheels against the inner rail with a tendency to cause derailment or perhaps a stalling of the train. To reduce this effect, the velocity for which the outer rail is superelevated should be taken less than the velocity for which the maximum centrifugal force is figured. A proper velocity for the superelevation is herein assumed to be one that will give a centrifugal force equal to one-half of that for the maximum velocity. This will give an excess of load on the inner rail when the train is standing still equal to that on the outer rail when it is moving at maximum speed. The velocity for figuring the superelevation will, therefore, be taken at seven-tenths (0.7) of the maximum velocity. The assumed maximum velocity, properly reduced for curvature, can be taken from the formula,

$$V = 60 - 2.5\,D; \qquad \text{[Eq. 1]}$$

and the velocity for figuring the superelevation, from the formula,

$$V = 42 - 1.75\,D. \qquad \text{[Eq. 2]}$$

In these equations, V equals the velocity of the train in miles per hour and D equals the degree of curve.

These velocities are given for a level track or one on a slight ascending or on a descending grade in the direction of the traffic. Where the latter operates against a heavy grade on a track carrying trains in one direction only, the maximum velocity there should be reduced. The engineer should use his own judgment when such a case arises. In fact, in no instance should one adopt the velocities previously specified without first weighing carefully all the facts of the case in hand, and determining whether those velocities are proper. At times one has to be governed by requirements specified by others; and in case they do not agree with his own views, he should endeavor to have them modified.

The superelevation of the outer rail is to be determined from the formula,

$$s = \frac{4\,V^2}{R}, \qquad \text{[Eq. 3]}$$

where s = the required superelevation in inches,
V = the velocity of the train in miles per hour, reduced for the degree of curve as previously noted, and
R = the radius of the curve in feet.

Equation (3) is derived on the assumption that the rails are spaced five (5) feet on centres, and that the centre of gravity of the train is five (5) feet above the base of rail. Fig. 8*a* gives values of s for any degree of curvature up to twenty (20) degrees, for velocities ranging from ten (10) to sixty (60) miles per hour. It also indicates the values of s when V varies in accordance with Equation 2.

Superelevation for tracks on curves of electric railways should be figured in the same manner as for steam railways, to which class of lines the previous discussion has largely referred. In general, for the regular surface traffic in cities no superelevation need be allowed. Especially is this true when the bridge carries also highway traffic and when it is necessary to pave the floor. If any superelevation at all is provided under such conditions, it should be limited to one inch or, preferably, to one-half inch. Special cases, however, may arise where a greater superelevation is desirable, but this should be left to the judgment of the engineer. On elevated and interurban electric railway structures, superelevation should always be provided.

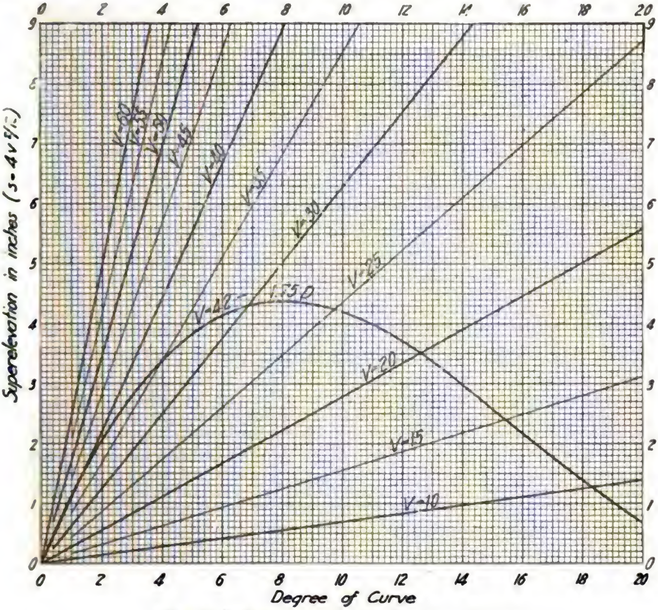


FIG. 8*a*. Superelevation for Tracks on Curves.

The centrifugal force is to be determined from the following formula:

$$F = \frac{WV^2}{15\ R} = cW, \qquad \text{[Eq. 4]}$$

where F = the centrifugal force in pounds acting horizontally at the centre of gravity of the load,
W = the moving load in pounds,
V = the velocity of train in miles per hour, reduced for the degree of curve as previously noted, and
R = the radius of curve in feet.

Fig. 8*b* gives values of the coefficient c, or $\frac{V^2}{15R}$, for any degree of curvature up to twenty (20) degrees and for velocities ranging from ten (10) to sixty (60) miles per hour. It also indicates the values of c when V varies in accordance with Equation 1.

No centrifugal force should be assumed to act on a bridge located on a curve for ordinary city surface lines, unless, in the opinion of the engineer, it is deemed advisable to do so. However, structures for interurban or rapid transit lines located on curves should always be figured for the effect of centrifugal force, the values given in Fig. 8*b* being used to determine the amount of such force.

Where wheel loads are adopted in computing the stresses in the various members of a structure, the percentages given by the curves in Fig. 8*b* are to be applied to these wheel loads in determining the centrifugal forces. Where equivalent uniform live loads are employed, these percentages are to be applied to them in the same way. In figuring stresses in the lateral bracing of bridges due to centrifugal force, the same equivalent uniform live loads are to be taken as are used for the trusses of such bridges; and in the case of stringer bracing (if it ever be computed at all), the equivalent uniform live load for centrifugal force should be that adopted for designing the stringers.

The centrifugal force should be assumed to act at a point five (5) feet above the base of rail, this being the average height of the centre of gravity of the live load.

Practically every important member in a structure is affected by the stresses produced by the centrifugal force, the superelevation of the outer rail, and the curvature of the track. The lateral bracing for the unloaded chords and the longitudinal bracing in tower bents are about the only members not so affected. Of course, the various members are by no means acted upon to the same extent; and in some instances the influence of some or all of the above factors can be neglected altogether. In general, however, they should be considered, or, at least, it should be ascertained that they are neglible.

As the effect of all the factors above noted, except that of transferring the centrifugal load to the ends of the structure, is merely to vary the distribution of the vertical loads on certain twin members, it would be possible so to arrange those members that the vertical loads on them would be uniform throughout their length. This is not practicable, however, as it would require the curving of every member parallel to the track. It is possible, though, so to adjust any pair of members with respect to the track that the vertical loads on them will produce the same quantitative effect in either moment or shear at a given point in either member. The effect of the transferring of the centrifugal load to the ends of the structure cannot be balanced in the same way, although it is possible to make an adjustment that will be advantageous under certain circumstances, as will be shown later.

When the outer rail is not superelevated and the centrifugal force is not assumed to act, only the effect of track curvature need be taken into

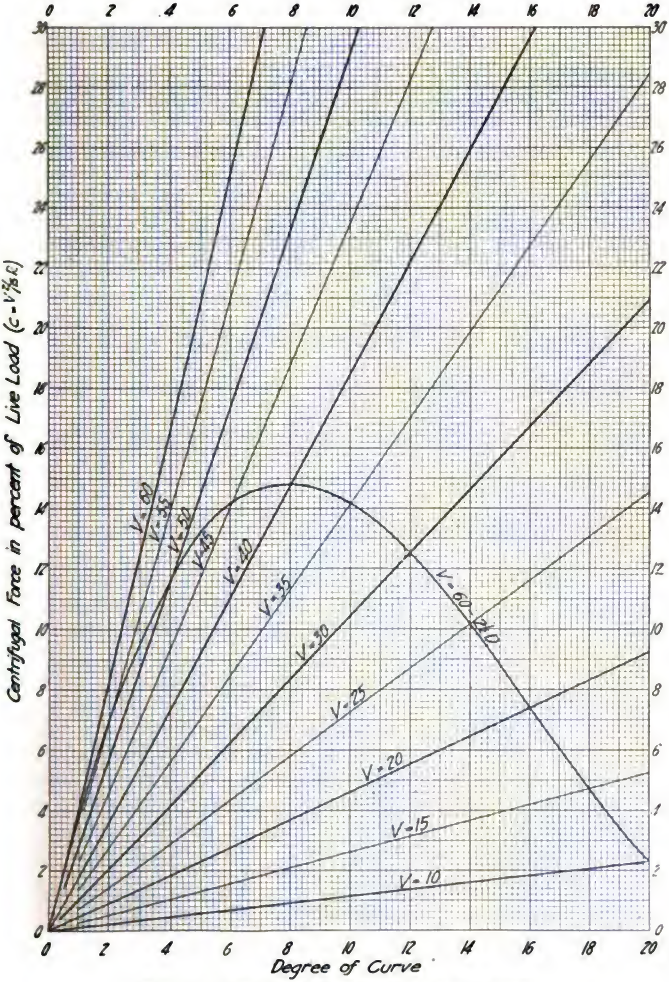


FIG. 8b. Centrifugal Force for Tracks on Curves.

consideration. The adjustment of the track for the effect of curvature, so as to produce the same moment or shear at a given point in twin members will depend on the location of the said point, as well as upon whether the moment or the shear is to be determined.

Let us consider two members, AB and CD in Fig. 8c, with their centre lines a distance d apart, carrying a curved track with a mid-ordinate m.

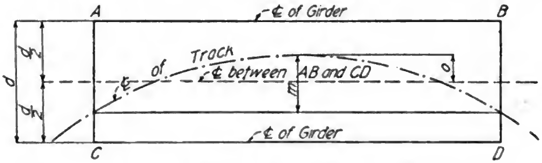


FIG. 8c. Layout of Girders on Curves

The long chord is assumed parallel to the two members. For equal moments at the centres of AB and CD, the offset o should equal $\frac{m}{6}$; for equal moments at the quarter points, it should equal $\frac{5m}{24}$; and for equal end reactions, it should equal $\frac{m}{3}$. It is possible to arrange the stringers, girders, and trusses to meet any one of the above named conditions, but it is not always advisable. In some cases it makes little difference which one of the above offsets is used or whether the offset is made equal to $\frac{m}{2}$ or even to m itself. Again, it may be deemed best to adopt some other fractional part of m as the offset. In any case, no matter how the centre line between any twin members is located with respect to the track, the eccentricity of the members from a balanced condition can readily be figured and the increase or decrease in moments and shears determined. From these it is possible to figure the extent to which such increases or decreases should be taken into account. For instance, if the centre line between the two members AB and CD is offset from the centre line of track at mid-span an amount equal to $\frac{m}{6}$, which arrangement gives equal moments at the centre, the moment on the inside girder is increased and the moment on the outside girder is decreased from the average moment M at the quarter point by the amount, $\left(\frac{5m}{24} - \frac{m}{6}\right) \frac{2}{d} M = \frac{mM}{12d}$, or by the percentage $\frac{8.3m}{d}$. Under the same condition the end reaction on the inside girder is increased and that on the outside girder is decreased from the average value R by the amount $\frac{mR}{3d}$, or by the percentage $\frac{33.3m}{d}$.

For stringers parallel to the chord of the curve, it will generally be satisfactory to make the offset from the centre of track to the centre line between stringers at centre of span a distance equal to $\frac{m}{2}$, and figure them for equal loads. The increase in moments and shears on the more heavily loaded stringer will be negligible. This same condition will also apply to girder spans up to about thirty (30) feet in length for sharp curves, and to greater lengths for flat curves. However, for greater lengths than thirty (30) feet it is difficult to set any definite limits, as the flange sections increase so rapidly. Each case should, therefore, be investigated on its own merits. Wherever it is not advisable to locate the centre line between girders at a distance of $\frac{m}{2}$ from centre line of track at mid-span, this offset should preferably be made $\frac{m}{6}$. This will give equal moments at the centre, although unequal end reactions. But with the end reactions differing considerably, the shop work will not be affected appreciably, as it will require merely the use of somewhat thicker end angles for the heavier reaction. Where the mid-ordinate for a curve on a stringer span is not greater than one (1) inch, the centre line between the stringers may be placed on the chord of the centre line of track, as the increases in the sections of the more heavily loaded stringer are negligible. For the stringer concentrations on the floor-beams and for the floor-beam reactions on the girders or trusses, the average of the two offsets for which the stringers in the panels adjacent to the floor-beam in question were figured should be used as the point at which the track load will be assumed to act.

If we call the point at which the track load is assumed to be applied x, the eccentricity E of the said load employed in developing the equations to follow will be taken as the eccentricity of the point x with respect to the centre line between stringers, girders, or trusses, and will be considered as positive when measured toward the stringer, girder, truss, or end of floor-beam in question, and negative when measured in the opposite direction. In all cases the eccentricity E will carry its own sign. These equations will be determined from the standpoint of a single-track structure, but they are equally applicable to multiple-track structures.

The stringers in a structure on a curve may be placed parallel to and symmetrically about the centre line of the bridge; they may be parallel to the said centre line, but offset therefrom so as to give equal moments at their mid-points; or they may be placed parallel to the chords of the curve between the floor-beams, and offset so as to give equal moments at mid-length.

In the first layout the eccentricity of the track will have to be considered. The moments can be computed for the average eccentricity, *i.e.*, the eccentricity of the middle point of the mid-ordinate to the curve between the floor-beams; and for flat curves the same eccentricity can be used for the end shears. However, for sharp curves, the inclination of the long chord of the track to the axis of the bridge must be taken into account in determining the end shears of the stringers, especially in the end panels. As the loads on the stringers vary, their sections will not be alike except when the differences are so small that they may be neglected. The end connections should all be designed for the maximum stringer end shear occurring in the span and for the maximum concentration on the floor-beams. This will make all floor-beams alike as to the stringer connections, although their flange sections and their end connections will vary. This layout of stringers will give the best arrangement for the shops, as practically the only extra work entailed will be the caring for the various flange sections and the details affected by them.

If we let W equal the total load on the two stringers, E the eccentricity, and b the distance from centre to centre of stringers, the load Q on either stringer will be

$$Q = \frac{W}{2}\left(1 + \frac{2E}{b}\right). \qquad \text{[Eq. 5]}$$

In this equation E carries its own sign. Where the chord of the curve is appreciably inclined to the axis of the stringers, this equation cannot be used for determining the end shears.

In the second layout the stringers will be figured for equal moments at the centre, and the end connections for equal shears where the curve is flat. However, where the curve is sharp (and sometimes in the end panels for flat curves) due consideration must be given to the inclination of the chord of the curve between the floor-beams to the axis of the stringer when determining the end shears. The end connections should always be designed for the maximum end shear. In this case the sections of all the stringers will be alike, but the end connections will be different on account of the offsetting. This requires the outstanding legs of the end connections to vary in order to engage the same holes through the floor-beams. For the same reason the stringer connections will not be symmetrical about the centre of the floor-beams, causing extra shop work on the latter.

In the third layout the stringers will be figured for equal moments and equal end shears. In this case the stringers will have the same sections and end connections, but the lengths of the stringers will vary; and the end connections as well as the flanges will have to be bevelled. The bevels may be so large that bent plates will have to be employed instead of connection angles; especially will this be true in the end panels. Moreover, the stringer connections will not be symmetrical about the centre line of floor-beams. This last case involves the worst condition for shop work, although it effects the best distribution of the loads to the stringers.

Before adopting one of the three methods outlined for laying out the stringers, the case in hand should be carefully studied. In general, it might be said that for very flat curves the first method will prove the most satisfactory; for medium curves, the second method; and for sharp curves, the last method. However, even this consideration may be affected by the length of the span as well as by that of the panels.

In designing the floor-beams, two cases may arise—one in which the stringers are parallel to the trusses and spaced symmetrically about their centre line, and the other in which the stringers are parallel either to the trusses or to the chords of the curve between the floor-beams and offset from the centre line between the trusses so as to follow the curve as nearly as possible. In either case it is best first to find the end reactions of the floor-beams on the two trusses. This can be done most readily by determining the track concentrations on the floor-beams in the case of either concentrated or uniform loads for two panels, considering the track straight, and then computing the truss loads by using the eccentricities of the said concentrations. If W_1 is the total track concentration on the floor-beam, E_1 the eccentricity, b_1 the distance from centre to centre of trusses, and R_1 the floor-beam reaction for either truss, then

$$R_1 = \frac{W_1}{2}\left(1 + \frac{2E_1}{b_1}\right), \qquad \text{[Eq. 6]}$$

in which E_1 carries its own sign. After all the floor-beam reactions R_1 are found, the moment at any stringer point of a floor-beam can be readily determined. Where the stringers are spaced symmetrically about the centre line between trusses, to determine the moment for any floor-beam, it is only necessary to use the larger end reaction of the said floor-beam, as the moment arms are equal. However, where the stringers follow the track, the moments at all the stringer points will have to be determined, because the lever arms all vary. It may be possible to tell by inspection where the maximum moment in any floor-beam will occur. As it will also be necessary to know the stringer concentrations on the floor-beams, these can be determined from the equation given above by substituting R for R_1, and b for b_1, where R is the stringer concentration, and b is the distance from centre to centre of stringers. Therefore

$$R = \frac{W_1}{2}\left(1 + \frac{2E_1}{b}\right), \qquad \text{[Eq. 7]}$$

in which E_1 carries its own sign. When E_1 is zero, *i.e.*, when the stringers follow the track,

$$R = \frac{W_1}{2}. \qquad \text{[Eq. 8]}$$

The panel concentrations on the trusses are determined in the same way as are the floor-beam reactions.

For sharp curves, where the stringers are parallel to the axis of the bridge, it may be necessary to figure the floor-beam concentrations for each stringer separately before determining the floor-beam reactions or truss loads, on account of the difference in the inclination of the chords of the curve in the two adjacent panels. Where this difference is appreciable, the averaging of the eccentricities in the adjacent panels will not be sufficiently accurate, and all floor-beam concentrations computed in this manner will be excessive. The greatest difference will show up in the end floor-beams and will readily indicate to what extent the figuring by average eccentricities will affect the floor-beam concentrations and reactions and the truss panel loads.

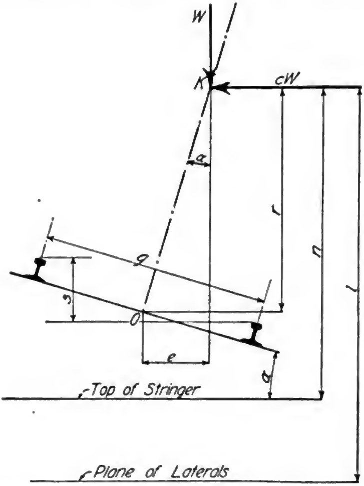


FIG. 8*d*. Forces Acting on a Structure on a Curve.

In the preceding treatment of the subject, consideration has been given only to the effects of track curvature. This is as far as it is necessary to carry the investigations in case the superelevation is omitted and the centrifugal force is ignored. However, where superelevation is provided and the centrifugal force is taken into account, due consideration must be given to the effects thereof.

In all previous notations the impact factor has been ignored, and all loads must either be increased for this or else be assumed to include it. In figuring the effects of the centrifugal force and superelevation, however, no impact is to be added. For this reason an impact factor will be applied hereafter to all loads affected by impact. All equations will hold true for the effects of track curvature alone, if all factors due to centrifugal force and superelevation are cancelled.

The effect of the centrifugal force is a tendency to overturn the train, thereby increasing the load on the outside rail and decreasing that on the inside one by an amount equal to the overturning moment divided by the distance from centre to centre of rails. By superelevating the outer rail the centre of gravity of the train is shifted toward the centre of the curve, thus producing a negative moment to counteract all or part of the overturning moment from the centrifugal force. The resultant effect is to vary the distribution of the vertical loads on the stringers, floor-beams, and girders or trusses.

In Fig. 8*d* let the plane of the rails make an angle a with the horizontal. Sin α then equals $\frac{s}{g}$, where s is the superelevation of the outer rail and g the distance from centre to centre of rails. The centrifugal force cW is applied at K, the centre of gravity of the mass, located at a distance r above the base of rail. The distance r should actually be taken along OK; but the angle α is so small that the vertical and the inclined distances are not appreciably different. Assume K to be located at a distance n above the tops of the stringers and at a distance l above the plane of the laterals. The load W is eccentric with respect to O, the central point between rails, by an amount $e = r \tan \alpha$. As the angle a is small, we can write $\sin \alpha$ in place of $\tan \alpha$, it being more convenient to do so as $\sin \alpha$ is known. Then, the increase in load on the outer rail or the decrease in load on the inner rail is

$$S_2 = \frac{W}{g}(cr - e) = \frac{Wr}{g}(c - \sin \alpha). \qquad \text{[Eq. 9]}$$

In the same way the increase in load on the outer stringer or the decrease in load on the inner stringer is,

$$S = \frac{W}{b}(cn - r \sin \alpha); \qquad \text{[Eq. 10]}$$

and for the trusses the corresponding formula is

$$S_1 = \frac{W}{b_1}(cl - r \sin \alpha). \qquad \text{[Eq. 11]}$$

If we combine the effects of track curvature, superelevation, and centrifugal force, the following equations will result, using the same notation as given before and letting I equal the impact coefficient.

For stringers parallel to and symmetrical about the centre line of bridge, the loads on the stringer are:

Outside stringer,

$$Q = W\left\{(1 + I)\left(\tfrac{1}{2} + \frac{E}{b}\right) - \frac{r \sin \alpha}{b} + \frac{cn}{b}\right\}, \qquad \text{[Eq. 12]}$$

Inside stringer,

$$Q = W\left\{(1 + I)\left(\tfrac{1}{2} + \frac{E}{b}\right) + \frac{r \sin \alpha}{b} - \frac{cn}{b}\right\}. \qquad \text{[Eq. 13]}$$

In these equations E carries its own sign.

For stringers following the curve with no appreciable eccentricity, these loads are:

Outside stringer,

$$Q = W\left\{\tfrac{1}{2}(1 + I) - \frac{r \sin \alpha}{b} + \frac{cn}{b}\right\}, \qquad \text{[Eq. 14]}$$

Inside stringer,

$$Q = W\left\{\tfrac{1}{2}(1 + I) + \frac{r \sin \alpha}{b} - \frac{cn}{b}\right\}. \qquad \text{[Eq. 15]}$$

To produce equal loads on the two stringers, the eccentricity E, should be

$$E = \frac{1}{1 + I}(r \sin \alpha - cn), \qquad \text{[Eq. 16]}$$

the sign being taken with reference to the outside stringer. This gives a load of $\frac{W}{2}(1 + I)$ on each stringer. The load on the inside stringer for the train standing still and no impact considered will never be as large as this.

After determining the coefficients in the parentheses for the case in hand, as well as the centre moment and the end shear for the total load on the two stringers, the actual moment and shear in any stringer can be found by multiplying the above calculated moment and shear by the coefficient for this stringer. As stated before, in computing the end shears for stringers, the effect of the inclination of the chord of the curve to the axis of the stringer should be looked into carefully.

The above equations given for stringers can also be applied for similar conditions to deck plate girder and deck truss spans without floor systems. However, it will be necessary to investigate other sections than merely the centre and end ones in order to prevent any appreciable overstress. The said overstress can be determined as previously described; and if necessary, the sections can be increased.

For stringers parallel to and symmetrical about the axis of the bridge, the stringer concentrations on the floor-beams are:

Outside stringer,

$$R = W_1\left\{(1 + I)\left(\tfrac{1}{2} + \frac{E_1}{b}\right) - \frac{r \sin \alpha}{b} + \frac{cn}{b}\right\}, \qquad \text{[Eq. 17]}$$

Inside stringer,

$$R = W_1\left\{(1 + I)\left(\tfrac{1}{2} + \frac{E_1}{b}\right) + \frac{r \sin \alpha}{b} - \frac{cn}{b}\right\}. \qquad \text{[Eq. 18]}$$

In these equations E_1 carries its own sign.

When the stringers follow the curve with no appreciable eccentricity, these concentrations are:

Outside stringer,

$$R = W_1 \left\{ \tfrac{1}{2}(1+I) - \frac{r \sin \alpha}{b} + \frac{cn}{b} \right\}, \qquad \text{[Eq. 19]}$$

Inside stringer,

$$R = W_1 \left\{ \tfrac{1}{2}(1+I) + \frac{r \sin \alpha}{b} - \frac{cn}{b} \right\}. \qquad \text{[Eq. 20]}$$

After determining the coefficients for all floor-beams as well as the total track concentration on one floor-beam, the actual concentration at any stringer point can be found by multiplying the figured track concentration by the coefficient for this point.

For floor-beams, the end reactions on the trusses are:

Outside truss,

$$R_1 = W_1 \left\{ (1+I)\left(\tfrac{1}{2} + \frac{E_1}{b_1}\right) - \frac{r \sin \alpha}{b_1} + \frac{cl}{b_1} \right\}, \qquad \text{[Eq. 21]}$$

Inside truss,

$$R_1 = W_1 \left\{ (1+I)\left(\tfrac{1}{2} + \frac{E_1}{b_1}\right) + \frac{r \sin \alpha}{b_1} - \frac{cl}{b_1} \right\}. \qquad \text{[Eq. 22]}$$

These coefficients can be determined for all the floor-beams and also the total load on one floor-beam. The actual reaction for any floor-beam at either truss can then be found by multiplying the total floor-beam load by the coefficient for the end in question; and the moments at all stringer points can consequently be computed.

The panel loads for the trusses can be determined from the equations for the floor-beam reactions.

In addition to varying the vertical loads on stringers, girders, and trusses, the centrifugal load produces stresses in the loaded flanges of stringers and girders and in the loaded chords of trusses due to the transferring of this load horizontally to the ends of the span. Tension is produced in the outside, and compression in the inside flange or chord. The said load in a stringer or a deck-girder span is taken to the ends by the lateral bracing and then to the supports through the cross frames; while in a span with a floor system, it is first transferred by the stringer bracing to the floor-beams and then to the ends of the span through the lateral bracing.

In a deck span the stresses in the outside loaded chord are decreased and those in the inside loaded chord are increased by the transferring of the centrifugal load to the ends of the span. It is impossible to shift the stringers, girders, or trusses, with respect to the track, so as to equalize the stresses in all the flanges or chords, because the unloaded flanges or chords are not affected by the horizontal action of the centrifugal load. If the stresses in all the flanges or chords are equalized for all other loads than the one just mentioned, any attempt to balance the loaded flanges or chords will unbalance the unloaded flanges or chords. In stringer and girder spans, in which all flanges are made up of the same style of section, the span can be shifted to advantage toward the inside of the curve until the stress in the top flange of the inside girder equals that in the bottom flange of the outside girder and *vice versa*. This will give the minimum variation in flange sections, as well as the least number of such differing sections. To obtain this result. the track will have to be shifted an amount $\frac{c\,d}{2\,(1 + I)}$, in which c is the ratio of the centrifugal to. the direct load, d is the depth of the stringer or girder from centre to centre of flanges, and I is the impact coefficient. Where the difference in the flange stresses is slight, as in stringer spans, all sections can be made alike.

In truss spans and in deck, plate-girder spans having top flanges which are built up of four angles and bottom flanges which are composed of two angles and plates, it is best not to attempt to equalize the stresses of the diagonally situated flanges or chords, as the make-ups of the said top and bottom flanges or chords are different, and hence no benefit would result from such a change. It will be best to equalize the stresses in the unloaded flanges or chords and let the loaded flanges or chords take care of themselves. This will at least give the same sections for the unloaded flanges or chords.

In trestles on curves the towers and bents must be figured for the shear and overturning due to the centrifugal load in the same manner as for wind loads.

If the stringers or girders are inclined the same amount as the rails, due consideration must be given to this condition, as it tends to equalize the loads without shifting the track to the same extent as is done for vertical stringers or girders.

In all the preceding investigations it has been assumed that the centrifugal load is normal to the axis of the bridge throughout its length. Theoretically this is not correct, as the centrifugal load acts radially at any point. However, the difference between the radial load and its normal component is not appreciable; and assuming the former to be normal to the axis of the bridge involves a slight error on the side of safety.

The centrifugal load is to be treated in the same manner as the live load and not as an unusual load, for it always exists when the live load is acting; consequently there will be no increase in permissible unit stresses when determining the sections of members affected by the centrifugal load, unless there be wind load or other unusual loading in the combination. In other words, the centrifugal load *per se* does not warrant the increasing of the intensities of working stresses.

While the preceding discussion assumes that the location of the track with respect to the steel work may be varied as desired, it is not always practicable or economical to do so; and especially is this true in through spans of either the girder or the truss type. The girders or trusses may be spaced such a distance apart that the position of the track can be arranged so as to fulfill the foregoing requirements as to equalizing the loads on them as far as it is possible to do so, and in addition to provide the required clearance at the inside truss. However, this is likely to give an excessive distance from centre to centre of trusses. It might be more economical to reduce this distance, merely providing the required clearance for both the inside and the outside trusses and taking care of the unequalized loads on them. This will in general make the shop work of the two trusses or girders (especially that of the trusses) vary to a greater extent than otherwise, because for an equalized condition it will usually be possible to design the trusses very nearly alike. In either case, however, the total weight of the trusses will be about the same, as the shifting of the tracks merely changes the division of loads on them. With the trusses or girders placed farther apart than is required for clearance, the weight of the floor system and lateral bracing increases; consequently the question as to which of the two arrangements will prove the more economical must be determined by comparing the cost of the extra metal in the floor system of the wider span with the cost of the extra shop work in the narrow span. In general, though, the most economical arrangement will be to make the distance from centre to centre of trusses as small as practicable.

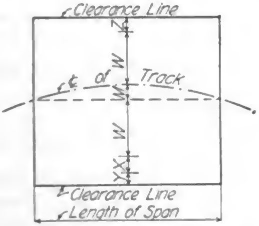


FIG. 8e. Clearance Diagram for Square, Through Bridges on Curves.

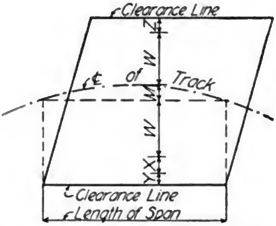


FIG. 8f. Clearance Diagram for Skew, Through Bridges on Curves.

To provide the same clearance for single-track through-bridges on curves as required on tangent, the clear width between trusses is to be as shown in Figs. 8e and 8f.

In these figures,

W = the lateral clearance from the centre line of track required for tangent alignment,

M = the mid-ordinate of the curve for a chord equal to the span length,

X = an addition for the overhang of the centre of the car on the inside,

Z = an addition for the overhang of the end of the car on the outside, and

Y = an addition for the tilting of the car due to the superelevation of the outer rail.

X is the mid-ordinate of a chord of the curve the length of which is equal to the distance from centre to centre of trucks, whereas X plus Z is the mid-ordinate of a chord of the curve the length of which is equal to the length of the car. Where the distance from centre to centre of trucks is seven-tenths (0.7) of the length of the car, X equals Z; where this distance is greater than seven-tenths (0.7) of the length of the car, X is greater than Z; and where less than seven-tenths (0.7) of the length of the car, X is less than Z. Where X and Z are not equal, the total overhang should be assumed as twice the larger, or else due account should be taken of the fact that they are not equal, if the difference is appreciable. Unless otherwise specified, the clearance should be figured for a car eighty-five (85) feet long and sixty (60) feet between truck centres. In this case X will equal Z, and will be one (1) inch for each degree of curvature.

Y is to be taken equal to $\frac{sh}{5}$, where s is the superelevation of the outer rail in inches and h equals the greatest height of the car in feet above base of rail. In no case should it be assumed greater than fifteen (15) feet, making the maximum value of Y equal to $3s$. The effect of the tilting of the car is taken into account on the inside of the curve only. On the outside thereof the clearance is really increased by the tilting, but this increase is so small (usually less than one inch) that it should be neglected.

When the outer rail is not superelevated, Y becomes zero. When Y is zero and X equals Z, the centre line of the span bisects the mid-ordinate of the curve for a chord equal to the span length. This is true in both square and skew spans. In a skew span, however, it should be noted that the loads on the two girders or trusses are not balanced as they are in a square span. The offsets from the inside girder to the curve at its ends are equal, thus producing balanced loads about the centre; however, for the outside girder these offsets are different, the one at the acute angle being larger than that at the obtuse angle—the difference depending on the amount of skew—thus giving unsymmetrical loads on the girder. The moments and shears on the inside girder can, therefore, be figured the same as for a square span. However, the outside girder should be computed for loads from the part of the curve between its ends, if the difference in the offsets is of sufficient importance to warrant such a consideration.

CHAPTER IX

WIND LOADS, VIBRATION LOADS, AND TRACTION LOADS

THE wind pressure per square foot of exposed surface for which bridges should be designed has always been an unsettled matter. Many experiments looking toward its solution have been made, but the force in question is such a variable one, and is so greatly influenced by many factors which are difficult to control, that the results are not all accordant. It is generally agreed, however, that the pressure on any particular surface varies with the square of the velocity of the wind, and that the intensity diminishes as the area acted upon increases. In general, the intensity on a plane surface, normal to the direction of the wind, can be expressed by the formula,

$$P = KV^2, \qquad \text{[Eq. 1]}$$

where P is the pressure in pounds per square foot, and V is the velocity in miles per hour. The value of K cannot be given with any certainty, but is generally considered to lie between 0.003 and 0.005, most of the later writers assuming it as 0.004 or less. In the 1910 edition of "Modern Framed Structures" the values 0.0032 and 0.004 are both given, the smaller figure being considered the more likely of the two; but some other authorities prefer the higher figure. For a velocity of one hundred (100) miles per hour, which rarely occurs in the United States except in tornadoes, the resulting pressures from the two values given above are thirty-two (32) and forty (40) pounds per square foot. It is evident, therefore, that the highest probable unit pressure that will be developed, on a bridge in ordinary localities will lie between thirty (30) and forty (40) pounds per square foot. As the wind is rarely uniform over any extended area, the average pressure on a span of any size will be considerably below these figures. Furthermore, a wind pressure in excess of thirty (30) pounds would overturn high, empty box cars, so that a greater pressure than this would never be assumed to act in conjunction with the live load.

Specifications have frequently called for unit pressures as high as fifty (50) pounds per square foot on the unloaded structure, and from thirty (30) to forty (40) pounds on the loaded structure. These intensities have generally been used in designing the main members of the lateral system; but pressures of half the amounts would almost always have caused failure in the connections employed for the lateral members, and, in pin-connected single track spans, would have buckled the bottom chords when the structure was empty. C. Shaler Smith, the noted bridge engineer, after an extended examination of existing bridges, came to the conclusion that it was very doubtful if bridges other than those of short span ever had to withstand pressures much in excess of thirty (30) pounds per square foot. It is believed, therefore, that the following intensities provide amply for all probable effects of wind, although in certain localities it might be thought advisable to increase the pressures for the unloaded structures.

First. For the unloaded structure, pressures per square foot of about thirty-five (35) pounds for spans two hundred (200) feet long, and thence decreasing to thirty (30) pounds for spans six hundred (600) feet long, and to twenty-five (25) pounds for spans one thousand (1,000) feet or more in length.

Second. For the loaded structure, a pressure of thirty (30) pounds per square foot for spans two hundred (200) feet or less in length, ranging down to twenty-five (25) pounds for spans six hundred (600) feet long, and to twenty (20) pounds for spans one thousand (1,000) feet or more in length.

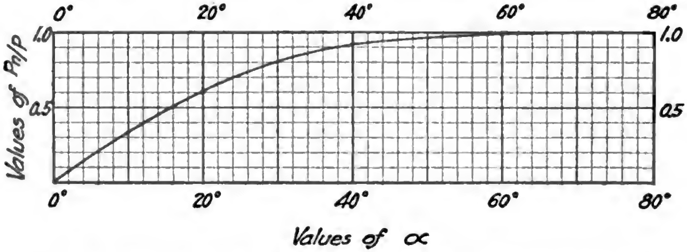


FIG. 9a. Wind Pressures on Inclined Surfaces per Duchemin's Formula.

For bridges carrying highway traffic only, the live load and the wind load will not be considered to act simultaneously, for the reason that no person would ever venture upon the structure when there exists a wind pressure per square foot of anything like thirty (30) pounds.

The intensities above discussed have referred entirely to pressures on surfaces normal to the direction of the wind. It is frequently necessary to know the amount of pressure on surfaces which make an oblique angle with its direction. The formula most generally employed for this purpose is Duchemin's, which is

$$P_n = P \frac{2 \sin \alpha}{1 + \sin^2 \alpha}, \qquad \text{[Eq. 2]}$$

where α = Angle between the surface and the direction of the wind,

P = Pressure per square foot on a surface normal to the direction of the wind,

and P_n = Normal component of this pressure.

The curve in Fig. 9a gives ratios of P_n to P for various values of α.

In Fig. 9b are given the author's specified wind pressures and vibration

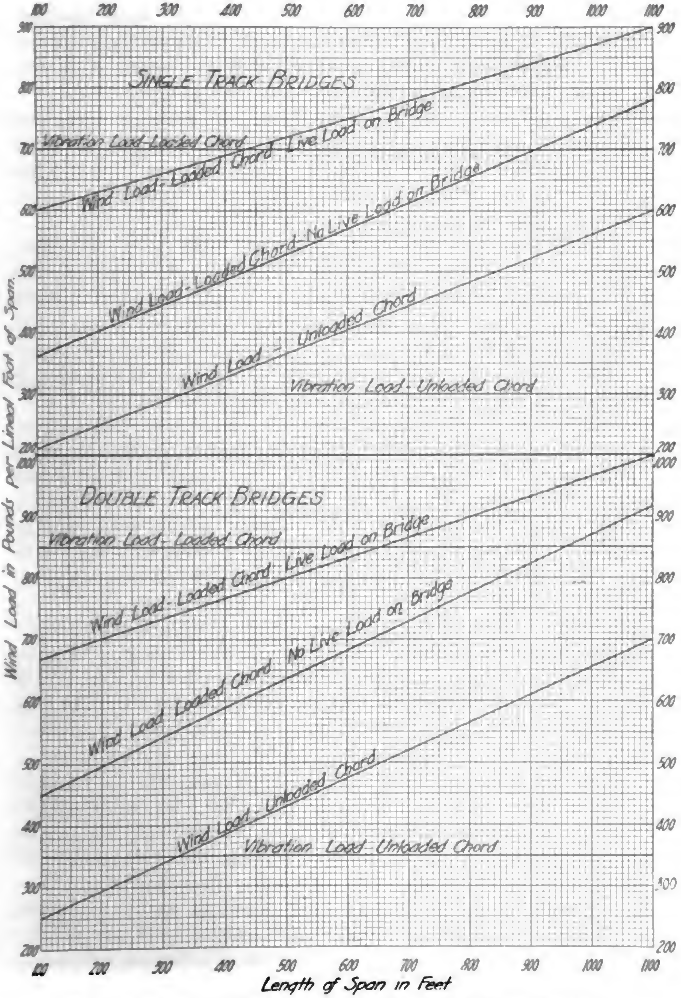


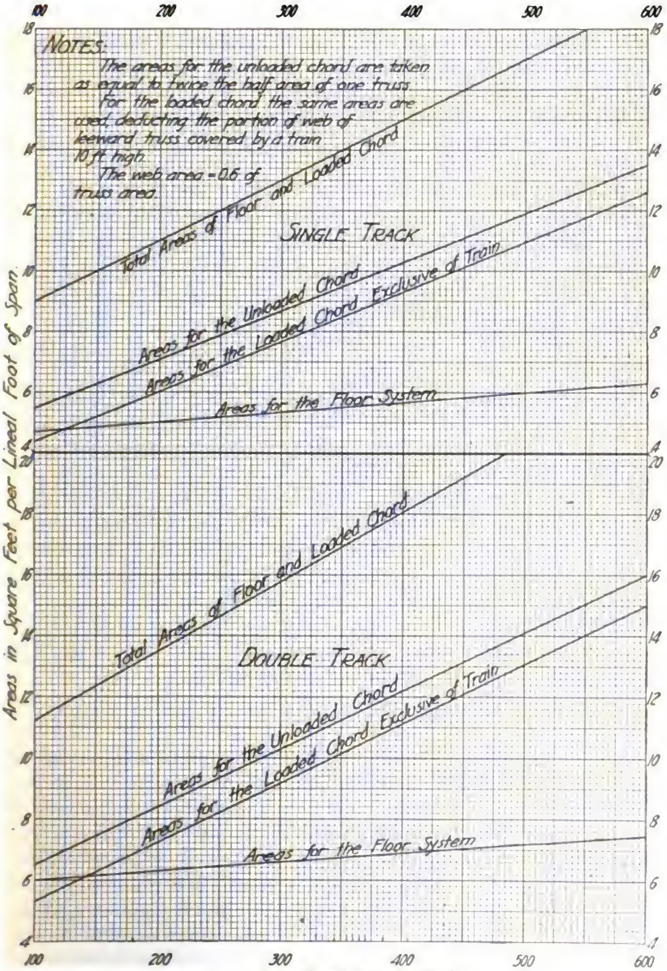
FIG. 9b. Wind Loads and Vibration Loads for Railway Bridges.

loads per lineal foot of span for the loaded and the unloaded chords of both single-track and double-track railway truss bridges. In Fig. 9*c* are shown the areas per lineal foot of span opposed to the wind by single-track and double-track railway bridges. From this last diagram the wind pressures per lineal foot of span given in Fig. 9*b* were computed by using the intensities previously noted.

It is really only in long-span railway bridges that the specified wind pressure cuts any figure; because, in spans of ordinary length, the vibration load (to be mentioned later) will govern the design of the lateral system; and modern live loads are so heavy that the combined live, impact, dead, and wind load stresses with the allowable increase in intensities of working stresses usually require less sectional area than the combined live, impact, and dead load stresses with the ordinary intensities, except where the wind produces transverse bending in a member, as in the end-posts of through bridges. While the wind load may reverse the stresses in certain members, this is of no importance in riveted structures, which are adopted almost exclusively for short spans; and even in pin-connected bridges the stiffening of the end sections of the bottom chords, which is specified to-day for all first-class structures of this type, will generally take care of any reversals that may occur. However, the wind stresses should always be figured for any bridge, unless the designer be very familiar with structures of just the type he has employed for the case in hand; and the test for wind combination should invariably be made, as it may cause a slight increase here and there in the sections of main members. The test for anchorage against overturning should always be made, particularly in the case of high trestles.

The wind loads for highway and electric railway truss bridges adopted in the specifications of Chapter LXXVIII are given in Fig. 9*d*. The curves of wind loading for Class A highway bridges are also to be used for electric railway spans when no live load is on the structure. The same curves are also to be employed when the live load is on the bridge, except that in no case should the wind load on the loaded chords be taken less than 480 lbs. per lineal foot of span. The diagram in Fig. 9*d* was computed for a clear roadway of twenty (20) feet. For wider structures the wind loads on the loaded chords are to be increased two (2) per cent for each foot of width in excess of twenty (20). For instance, a six hundred and forty (640) foot highway bridge of Class A has a clear roadway of thirty (30) feet between trusses and two exterior foot-walks each ten (10) feet wide in the clear—what are the proper wind loads to adopt? The diagram gives 470 lbs. per foot for the loaded chord and 310 lbs. per foot for the unloaded chord. These figures are to be increased by a percentage equal to $2 (30 + 2 \times 10 - 20) = 60$, making them respectively 750 and 500.

The proper wind loads to adopt for combined railway and highway bridges will have to be determined for each case as it arises by the use of trained judgment. A fairly good rule therefor is the following:



NOTES:

The areas for the unloaded chord are taken as equal to twice the half area of one truss.

For the loaded chord the same areas are used deducting the portion of web of leeward truss covered by a train 10 ft. high.

The web area = 0.6 of truss area.

FIG. 9c. Areas of Railway Bridges Exposed to Wind.

Find approximately the sum of the various live, impact, and dead loads for the combined bridge, and the corresponding sum for a similar railroad bridge without the highway attachment, and call the ratio of these sums r (greater than unity). The wind pressures per foot for both the loaded and the unloaded chords, given in Fig. 9*b*, should then be increased by a percentage equal to 25 $(r - 1)$.

The proper wind loads to use in designing railway, highway, and electric railway girder spans, trestles, viaducts, and elevated railroads are specified in Chapter LXXVIII. The proper loads for the design of movable bridges are also indicated therein. It will be noted that the maximum wind loads while the movable spans are being operated are taken only one-half as great as those assumed when the span is at rest. This is because no vessel would dare to navigate with a wind pressure greater than fifteen (15) pounds per square foot. For swing spans, it is specified that the machinery shall be capable of holding the span in any position against an unbalanced wind load of ten (10) pounds per square foot over the entire exposed area of one arm, also that it shall operate the span in the specified time against a similar unbalanced wind load of one pound per square foot. It is further provided that the span shall be effectively anchored (either by its own weight or by special detail) against the effect of an upward wind load of from ten (10) to fifteen (15) pounds per square foot on the total area of the horizontal projection of one arm of the span.

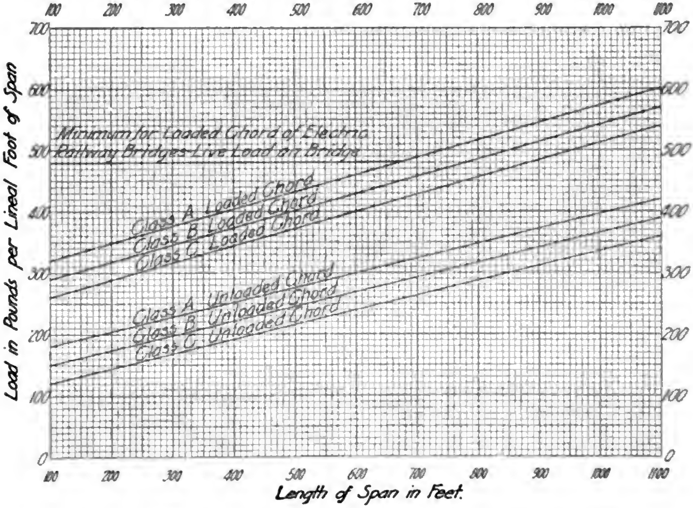


FIG. 9*d*. Wind Loads for Highway and Electric Railway Bridges.

For bascules, it is specified that the machinery shall be able to hold the span in any position against a wind pressure of fifteen (15) pounds per square foot, and to operate it in the specified time against a wind pressure of two (2) pounds per square foot, on the exposed area as seen in vertical projection. The vertical lift bridge is also required to operate in the specified time against a horizontal wind pressure of two (2) pounds per square foot, but the effect of this is very slight, being confined to the friction in the guides. There is probably a slight vertical wind pressure at times on the floor of a lift bridge; but it will have practically no effect on the time of operation, and as the maximum probable pressure during the action of the fifteen (15) pound wind specified above would not stress the operating ropes unduly, its existence can be entirely ignored. For all types of movable bridges an increase of thirty (30) per cent in the regularly specified unit stresses for main structural parts, and of one hundred (100) per cent for machinery parts, is permitted when the maximum wind pressure is acting.

The principal effects of wind loads on bridge structures are as follows:

First. They produce direct stresses in the members of the lateral systems and sway bracing.

Second. They produce direct stresses in the chords of the main trusses when they serve as the chords of the lateral system, as is usually the case.

Third. The transference of the wind pressure on the train down to the plane of the lateral system which carries it to the end of the span produces vertical reactions on the main trusses.

Fourth. The carrying of wind shears by the transverse strength of main members, as in the end posts of through bridges, causes bending moments in the said main members, and frequently in their connections or supports.

Fifth. The transference of wind shears by vertical or inclined bracing gives rise to transferred loads, which frequently produce stresses in the main truss members.

Sixth. They tend to overturn the structure as a whole.

Seventh. In movable spans, they put additional loads on the machinery.

The vibration load is a transverse loading, generally in excess of the wind load, which is applied solely to the lateral bracing of railway bridges. The stresses which it produces are not to be added to any other stresses, its sole object being to ensure sufficient sectional areas for lateral members in order to attain proper rigidity for the structure as a whole. For the loaded chords of through and deck spans, and for viaduct towers, its value is to be taken at seven hundred (700) pounds per lineal foot for single-track structures, and eight hundred and fifty (850) pounds per lineal foot for double track structures. For the unloaded chords the corresponding figures are, respectively, three hundred (300) and three hundred and fifty (350). Highway and electric railway bridges are not to be figured for vibration loads.

The "traction" load on bridges is the longitudinal force caused by the starting or stopping of a train or electric car. The stopping or braking effort is usually much in excess of the starting force. For ordinary railway traffic, the maximum braking effort that may be expected will be about twenty (20) per cent of the weight on the drivers of the locomotive, about ten (10) per cent of the weight of the fully loaded tender, about seventeen (17) per cent of the light weight of freight cars, and about twenty (20) per cent of the light weight of passenger cars. Empty freight cars will rarely weigh over twelve hundred (1,200) pounds per lineal foot, or empty passenger cars over two thousand (2,000) pounds per lineal foot. The total braking force exerted by a train will, therefore, be about fifteen (15) per cent of the weight of engine and loaded tender, plus a load per lineal foot of cars back of the tender amounting to about two hundred (200) pounds for freight trains and four hundred (400) pounds for passenger trains. As the uniform load assumed for a Class-50 loading is five thousand (5,000) pounds per lineal foot, the proper braking coefficient for this uniform load will be about four (4) per cent for freight cars and eight (8) per cent for passenger cars. For Class-40 loading the corresponding percentages are five (5) and ten (10). Since the heavy, double-header engines specified for the standard loadings are rarely used except with freight traffic, it is evident that the proper coefficient to be applied to the uniform load back of the engines is about five (5) per cent. For very short loaded lengths, therefore, the traction force should be assumed at twenty (20) per cent of the maximum live load; and this percentage should decrease to fifteen (15) per cent for spans about one hundred and twenty (120) feet long, and to ten (10) per cent for spans about two hundred and fifty (250) feet long. The specifications of Chapter LXXVIII provide that the percentage shall be determined by the formula,

$$T = \frac{4000}{140 + L}, \text{ with } T_{max} = 20 \text{ and } T_{min} = 10,$$

where T = percentage of total load to be used as the tractive force,
and L = loaded length in feet.

By this formula, for all lengths of sixty (60) feet or less T will equal twenty (20) per cent, and for all lengths of two hundred and sixty (260) feet or more T will equal ten (10) per cent. The curve in Fig. 9e gives the values of T for various loaded lengths.

For electric railway cars, the traction load should be assumed as twenty (20) per cent of the total load on the portion of the structure in question, irrespective of the loaded length, as there is little difference between the weights of full and empty cars.

The only portions of a truss bridge really affected by the traction load are the floor system, the lateral system adjacent to the loaded floor, and the chords near that floor; but the traction force should be transferred immediately from the stringers to the lateral system and from it to the main trusses without bending the floor-beams transversely. Its effect on the chords can generally be ignored because of its infrequent occurrence, unless some abnormal detailing should cause secondary stresses of undue magnitude; consequently, when proper detailing is used in the lateral system, as explained in Chapters XX, XXII, and LXXVIII, the effects of traction loading may be ignored in truss bridges. But in railway trestles it is quite another matter; for in them it is a most important factor, governing as it does the economic span length, because of its great effect on the longitudinal bracing of towers and on the column sections. In highway bridges and trestles no attention need be paid to traction loading, unless the structure carry also a street railway; for there is usually no way in which to produce from an ordinary highway live load any thrust worthy of consideration.

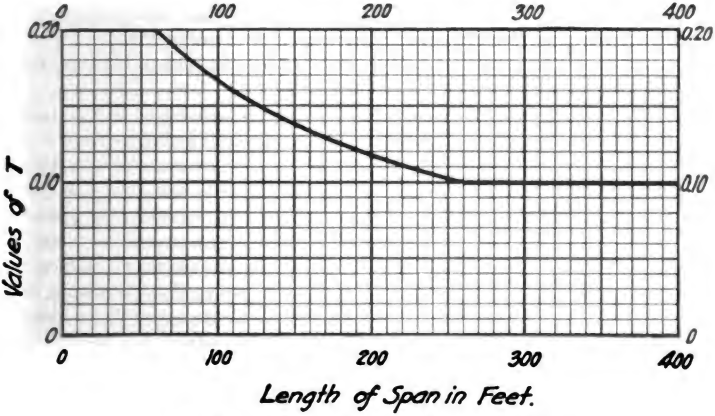


FIG. 9e. Traction Loads for Railway Bridges.

The effect on traction loading of connecting adjacent trestle-towers by riveting the intermediate girders to both of them is simply to reduce the average thrust per lineal foot of structure on account of the cars being lighter per lineal foot than the engines and tenders. Such an arrangement must not be assumed to halve the thrust moment by putting a plane of contraflexure near mid-height of tower; because to accomplish this the tops of the towers would have to be truly fixed by the longitudinal girders, a condition that a simple calculation will show to be impracticable.

CHAPTER X

METHODS OF STRESS COMPUTATION

It is not the intention in writing this chapter to handle the subject of stresses in the manner in which it is treated in the numerous text-books on bridges, for these can be referred to by any one desiring information concerning the mathematics of the subject; but simply to describe the various methods that are used by the engineering profession and to discuss their merits and demerits.

The oldest method of finding stresses in truss members is the analytic, and for trusses having equal panel lengths and parallel chords there is no other so simple and rapid; for with a table of coefficients, such as those given in this and other treatises, and by using a slide rule, it is the work of only a very few minutes to obtain the live-load stresses, the impact-load stresses, and the dead-load stresses for a span of any ordinary length, provided equivalent live loads and not wheel concentrations be employed. But when the panels are not of equal length or when the chords are not parallel, the graphic method is far superior to the analytic. The former method is an old one, in fact, nearly as old as the latter; but it has been enlarged and improved from time to time until now, as it is treated in the leading text-books, it is probably as perfect as it can well be made. It certainly is as clear and pretty a means of solving problems as one can imagine; and every student of engineering should be taught it so thoroughly that he will be able to use it just as readily as a clever schoolboy uses the four primary rules of arithmetic. European students of engineering are taught graphics much more thoroughly than are students in American technical schools; and, in the author's opinion, the curricula of the latter would be decidedly improved if their directors would follow the European lead in the treatment of this subject.

In Tables 10*c* to 10*f*, inclusive, at the end of this chapter, are given the coefficients for finding the live-load and dead-load stresses in chords and webs of Pratt truss and Triangular truss bridges having parallel chords and panels of uniform length; but the bridge computer should not be dependent on these tables, for he should be able to compute and record almost as rapidly as he can write them the coefficients for spans of these types having any number of panels. There is no need for expiaining how to make the computations, as the method is such a simple one; but there is one principle in it that the author has been using for thirty-five years, and which his assistants assure him is not covered in the text-books on bridges, viz., that the stress coefficient for any member of a chord for any position of loading is equal to the sum of the stress coefficients (for the same position of loading) of all the diagonals which attach to the chord between the member in question and the end of the span, provided that the shear does not change sign in that distance. Usually it is necessary to figure the chord stresses for the dead load only and to find those for the live load by proportion, using the slide rule.

The analytic method of computing stresses in trusses with polygonal chords, as given in the text-books, is lengthy and tedious. A far more simple method has been worked up of late years in the author's office by one of the computers who is not as much addicted as the author to the use of graphics; and in the hope that it may prove useful, it is here presented. Briefly stated, it is as follows:

First find the coefficient as if the chords were parallel, and record them. The stress in any chord member is then found by multiplying its coefficient by the ratio of length of the member to the length of the post in the same truss triangle and multiplying the product by the panel load. The total shear in any panel is the same as for a truss with parallel chords; but only a portion of this shear is carried by the web member, the inclined upper chord taking care of the remainder. The amount of shear carried by any chord member is equal to the vertical component of its stress, and can be found by dividing the said stress by the cosecant of its angle of inclination with the horizontal. To find the dead-load shear in any web member, it will be simplest to compute the shear thereon as for a truss with parallel chords, and then subtract from this figure the shear carried by the chord member in the same truss triangle. The same procedure is followed essentially in figuring the live-load stresses. As it would involve much extra work to calculate the chord stresses for the various positions of the live load, the shear in any member is found in the following manner: The total shear on the truss at the point in question is equal to the reaction R at the support ahead of the train. The horizontal component H of the stress in the chord member which carries part of this shear is

$$H = \frac{Rn_1p}{d},$$

where p is the panel length, d the length of the post in the same truss triangle as the chord member, and n_1 the number of panels from the support to the said post. The vertical component V of this chord stress, which is the shear carried by the chord member, is given by the formula,

$$V = \frac{Hy}{p} = \frac{Rn_1y}{d},$$

in which y is the difference between d and the length of the post at the other end of the chord member. The shear V_w on the web member itself is, therefore,

$$V_w = R \pm V = R\left(1 \pm \frac{n_1y}{d}\right) = R\left(\frac{d \pm n_1y}{d}\right)$$

The quantity R is the shear which would act on the member if the truss had parallel chords, which quantity is easily calculated. The quantity V_w is equal to the stress in a post, but must be multiplied by sec. θ to give the stress in a diagonal. Attention must be paid to the sign before the term n_1y. It is negative for main diagonal stresses and positive for counter-stresses.

The computation of stresses in a truss by the above method is carried through in the following manner: A diagram of the truss is first laid out, and the lengths of all the members are calculated and noted thereon. The stress coefficients are then determined as for a truss with parallel chords, and written on the various members. The value of sec θ is computed for each diagonal, and noted thereon. The ratio $\frac{\text{length of member}}{\text{length of post in same truss triangle}}$ is then figured for each chord member and also written thereon. There is next calculated for each top chord member the cosecant of its angle of inclination with the horizontal, and also the value of the expression $\frac{d \pm n_1 y}{d}$, both of which quantities are written above the member.

The panel dead loads are then computed. The upper and lower chord concentrations must be separated for the calculation of the post stresses, but their sum is to be used for all other members. The dead-load chord stress is then found for any member by multiplying this panel load by the product of the proper stress coefficient and the ratio, $\frac{\text{length of member}}{\text{length of post in same truss triangle}}$. The shears in the various web members are then computed as for a truss with parallel chords; and the shear in each member is reduced by the vertical component of the stress in the proper chord member, found as explained previously. The results give directly the post stresses; but they must be multiplied by the proper secants in order to obtain the stresses in the diagonals. The stress in the hip vertical, of course, is simply the load at its foot.

The panel live-load concentrations are next figured. The live-load chord stresses are then found by multiplying the dead-load stresses by the ratio of the panel live load to the panel dead load. The live-load post stresses are found by multiplying the panel load by the products of the stress coefficients and the ratios $\frac{d \pm n_1 y}{d}$ for the various members; and the live-load diagonal stresses by forming similar products, and then multiplying them by the proper secants.

In the early eighties there came into vogue the concentrated wheel-load method of finding stresses in bridge members; and while its establishment was certainly an important addition to engineering knowledge, its enforced use has, for the last thirty years, been a needless burden on bridge computers. This method very properly is taught in all the engineering schools, but those professors who are deficient in actual experience in computing bridges sometimes fail to point out how burdensome it is and to explain that the equivalent uniform load method gives results which agree closely enough for all practical purposes with the so-called exact method. These professors fail to recognize the fact the the wheel loads specified are merely typical, that in all probability no train with wheel spacings and loadings exactly or even very nearly coincident throughout with those of the assumed loading will ever pass over the bridge, and that, consequently, there is no need for hair-splitting exactness in finding the various stresses, especially when such refinement demands far more time than the equivalent method.

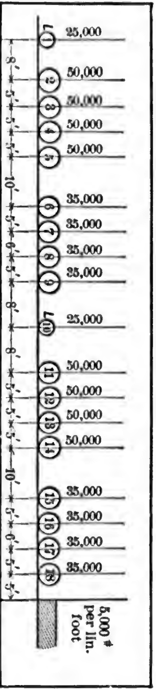
About the end of 1891 the author presented to the American Society of Civil Engineers a paper entitled "Some Disputed Points in Railway Bridge Designing," and obtained for it a most thorough discussion by more than forty members of the Society. In it he made a vigorous attack upon the "Concentrated Wheel Load Method" of computing stresses; and this point received much more attention in the discussion than any of the other points raised. After the publication of the paper he continued his investigations of this subject and published the results from time to time in the technical press, evoking considerable comment. The original paper, with the discussions, and all the subsequent papers have been collected and published by Mr. Harrington in "Principal Professional Papers." The reader who desires to study the history of this question and to assure himself of the correctness of the statement that the equivalent load method is practically as correct as the wheel concentration method is referred to that book.

As it behooves every bridge engineer to understand thoroughly the wheel concentration method of computing stresses, because by this the equivalent load curves are determined, there will now be given a concise but complete description of the said method and its operation. The first thing to be done with any newly established live loading of wheel concentrations is to prepare what is termed an "engine diagram," such as the one shown in Table 10*a* for Class 50 live load. To do this, the wheel loads and their spacings are laid out very accurately to some convenient scale, and vertical lines are drawn through the centers of the wheels and one through the head of the uniform load. The wheels are indicated by circles, those for the pilot wheels being small, those for the tender wheels larger, and those for the drivers still larger; and the uniform load is shown by a shallow shaded rectangle. The wheels are numbered inside of the circles, beginning with the pilot of the leading engine. Along the vertical line above each wheel is written the axle load thereof in pounds, and over the shaded rectangle is marked the uniform load in pounds per lineal foot of track. The wheel spacings are then recorded just below the wheels, thus completing the diagram portion of the table. In the top line of the table proper there is marked on the left of the vertical line through each wheel the distance of the said wheel from the pilot wheel of the leading engine; and in the second line there is registered on the right side of each vertical the distance of the wheel through which it is drawn from the head of the uniform load. In the third line there is noted in each space between consecutive verticals the sum of all the axle loads to the left; and in the fourth line there are similarly recorded the summations of the axle loads to the right of the various points. In the fifth line there is entered to the left of the vertical line through each wheel the moment of all the axle loads to the left about the said wheel. In the sixth line there are similarly shown the moments about any wheel of the various loads except No. 1 to the left thereof, and a heavy vertical line is drawn midway between Wheels 1 and 2. In the seventh line there are likewise registered the moments about each wheel of the various loads to the left except Nos. 1 and 2; a heavy vertical line is drawn midway between Wheels 2 and 3; and the moment of the axle load at 2 about Wheel No. 1 is written to the right of the vertical line through Wheel 1. To fill in the remaining lines of the table, the above process is continued, the heavy vertical line being drawn successively one space farther to the right. The number recorded in any horizontal line just to the left of the vertical through any wheel (in the portion above the heavy stepped line) will then represent the moment about the said wheel of the loads to the left over to the said heavy stepped line; while the number written in any horizontal line just to the right of the vertical through any wheel (in the portion below the heavy stepped line) will give the moment about the said wheel of the loads to the right over to the said heavy stepped line. As a check upon the work, the moments about Wheel 1 and about the head of the uniform load should be computed independently. In computing the various moments for an engine diagram the easiest method to adopt in finding the moment at any wheel is to add to the moment at the preceding wheel the product of the sum of the preceding loads by the horizontal distance of the said wheel from the preceding wheel.

In applying such a diagram, the truss to be figured is to be laid out by center lines to exactly the same scale as the diagram; and it must not be forgotten that blue-print paper shrinks nearly one per cent of both its length and its width in drying. Failure to recognize this shrinkage when using blue-print engine diagrams will cause errors that are often too large to be negligible.

The criterion for maximum shear on any web member of a truss with parallel chords and equal panel lengths is that the load on the panel must be made as nearly as possible equal to, but not greater than, the total load on the span divided by the number of panels in the span, it being understood that one of the wheel loads is to be placed exactly on the panel point at the end of the member to which the load is applied.

TABLE 10*a*. ENGINE DIAGRAM FOR CLASS 50 LOADING.



																			Summations of Loads for One Track in Thousands of Pounds		Distance of Any Wheel from Head of Uniform Load	Distance of Any Wheel from Wheel No. 1
																		0	730	0		0
38485	34775	31940	27915	24765	20765	17015	13515	10265	8840	7125	5585	4255	3100	1950	1060	400	0		705	25	111	
																	0	200				8
32845	29415	26160	23115	20245	16645	13295	10195	7345	6120	4085	3425	2375	1500	750	250	0			655	75	103	
																0	250	575				13
29570	26315	23235	20365	17670	14320	11220	8370	5770	4670	3410	2325	1450	750	250	0				605	125	98	
															0	250	750	1200				18
26545	23465	20560	17865	15345	12245	9395	6795	4445	3470	2385	1475	775	250	0					555	175	93	
														0	250	750	1500	2075				23
23770	20865	18185	15615	13270	10420	7820	5470	3370	2520	1610	875	350	0						505	225	88	
													0	500	1250	2250	3500	4325				33
18720	16165	13785	11615	9620	7270	5170	3320	1720	1120	560	175	0							470	260	78	
												0	175	925	1925	3175	4675	5625				38
16370	13990	11785	9790	7970	5870	4020	2420	1070	595	210	0								435	295	73	
											0	210	595	1645	2945	4495	6295	7395				44
13760	11590	9595	7810	6200	4400	2850	1550	500	175	0									400	330	67	
										0	175	560	1120	2420	3970	5770	7820	9045				49
11760	9765	7945	6335	4900	3350	2050	1000	200	0										365	365	62	
									0	280	735	1400	2240	3940	5890	8090	10540	11965				57
8840	7125	5585	4255	3100	1950	1050	400	0											340	390	54	
								0	200	760	1495	2440	3560	5660	8010	10610	13460	15085				65
6120	4685	3425	2375	1500	750	250	0												290	440	46	
							0	250	575	1310	2220	3310	4635	6085	9585	12435	15535	17285				70
4670	3410	2325	1450	750	250	0													240	490	41	
						0	250	750	1200	2110	3195	4490	5960	8560	11410	14510	17860	19735				75
3470	2385	1475	775	250	0														190	540	36	
					0	250	750	1500	2075	3160	4420	5890	7535	10385	13485	16835	20435	22435				80
2520	1610	875	350	0															140	590	31	
				0	500	1250	2250	3500	4325	5760	7370	9190	11185	14535	18135	21985	26085	28335				90
1120	560	175	0																105	625	21	
			0	175	925	1925	3175	4675	5625	7235	9020	11015	13185	16785	20635	24735	29085	31460				95
595	210	0																	70	660	16	
		0	210	595	1645	2945	4495	6295	7395	9215	11210	13415	15795	19695	23845	28245	32895	35420				101
175	0																		35	695	10	
	0	175	560	1120	2420	3970	5770	7820	9045	11040	13210	15590	18145	22295	26695	31345	36245	38895				106
0																			0	730	5	
0	175	525	1085	1820	3370	5170	7220	9520	10870	13040	15385	17940	20670	25070	29720	34620	39770	42545				111

Moments about any wheel of loads between wheel and stepped line.

For One Track–Two Rails

The corresponding criterion for any web member of a truss with chords not parallel is more complicated, for in that case the load on the panel must be made as nearly as possible equal to but not greater than the total load on the span divided by the product of the number of panels in the span and the expression $\left(1 + \frac{a}{b}\right)$, where a is the distance from the panel point considered to the end of the span ahead of the train, and b is the distance from the said end of span to the point of intersection of the produced top- and bottom-chord centre lines.

The criterion for maximum bending moment at any panel point of any simple truss is that the average load per lineal foot ahead of the said panel point must be as nearly as possible equal to, but not greater than, the average load per lineal foot on the whole span, it being understood that one of the wheel loads is to cover the panel point considered. It must be observed, however, that usually more than one position of the train will comply with this requirement; consequently it will be necessary to search for two or three such positions and adopt the one that proves to give the greatest bending moment.

While it is undoubtedly true that if a computer is provided with full tables of moments, floor-beam reactions, and end shears for a concentrated wheel load diagram, he can figure square, deck, plate-girder spans with about the same facility, and can find the stresses in square, equal-paneled trusses that have parallel chords by the expenditure of only two or three times the amount of mental energy required by the equivalent uniform load method; it cannot be denied that in designing half-through, plate-girder spans, skew bridges, trusses with polygonal top chords, trusses with subdivided panels, swing spans, cantilevers, arches, and suspension bridges, the work involved by the wheel-concentration method is enormously in excess of that required by the equivalent uniform-load method.

If one will take a Petit truss of, say, five hundred feet span, and having a polygonal top chord and the usual number of panels, and will compute the *truly greatest* live-load stress in every member thereof, he will find that the work will occupy many hours—possibly a day or more—while the corresponding stresses by the equivalent uniform-load method can be found in less than a single hour. Considering the fact that the results of the two computations will agree very closely indeed, that the assumed wheel loads and their spacings are merely typical and are not likely to be realized at all closely in railroad operation, that the value of the effect of impact is extremely uncertain and variable, and that in proportioning sectional areas of members a computer has to use plates and shapes of only approximately the total area needed, is it not evident that, for actual designing, the use of the tedious, hair-splitting, heart-breaking method of wheel computations is absurd? Each of the three just-mentioned uncertainties is liable to involve variations from exactness far greater than the differences between the equivalent uniform-load method and the so-called exact wheel-concentration method; hence it seems strange that there are to-day engineers and professors of civil engineering who adhere to the latter in the actual designing of bridges. Of course, there are reasons for this, and those who are in the business of designing and building bridges know them, even if the knowledge is not shared by the profession in general. The excuse for the professors is that they and their students generally confine their designing to the simplest kinds of structures, viz., square, deck, plate-girder spans, and square, Pratt-truss spans with parallel chords, in none of which the amount of extra labor involved by the wheel-concentration method is excessive. Again, professors naturally like to exemplify their theory by exact methods of computation, leaving to the students' future practice the acquisition of short cuts and labor-saving devices. In the case of the engineers of bridge companies, while they undoubtedly recognize the great time-saving qualities and the satisfactory exactness of the equivalent uniform-load method, they have for so many years been forced by the railroad companies to figure stresses by the wheel-concentration method that the habit of so doing has become second nature to them, hence their willingness to run along in the same old ruts year after year, wasting time, energy, and money in much useless figuring; for it must not be forgotten that the day of competitive bridge plans has not yet passed, and that consequently about eighty per cent of most bridge companies' computations go for naught. The blame for this wasted energy lies primarily with the railroad officials, who can understand easily what a train diagram means, but do not care to take the time or trouble to investigate sufficiently to convince themselves of the purely typical character of the assumed loading, how much it varies from actual conditions, the great variation in exactness in the assumed impact, the close agreement with pure theory that the equivalent uniform-load method affords, the great cost of making computations by the wheel-concentration method, and, finally, the fact that it is the railroad companies who eventually pay for the unnecessary expense. If these gentlemen were to give the subject a short but thorough investigation, the enforced use of the wheel-concentration method of figuring stresses would soon become a thing of the past; and it is to be hoped that the publication of this book will aid materially in overcoming this deplorable waste of time, brains, and money.

After any engine diagram has been adopted as a standard, in order to employ instead the equivalent uniform-load method the first thing to do is to prepare curves of total end shears and of equivalent uniform loads, such as those shown in Figs. 6*c*, 6*d*, and 6*e*. The end shears are required for plate-girder spans only, hence there is no necessity for carrying the computations beyond the limiting span-length for such structures, or, say, one hundred and ten (110) feet. The amounts are figured by placing the leading driving wheel at the end of each assumed span and finding the reaction there, then plotting the results on a diagram. The spans figured should vary in length by five (5) feet up to about forty (40) feet, and by ten (10) feet beyond that length.

There should be two diagrams of equivalent loads prepared, one for plate-girder spans and the other for truss spans. For the former the equivalents can be computed with sufficient accuracy by finding the maximum bending moment in foot-pounds at mid-span and substituting it for M in the formula,

$$w = \frac{8\,M}{l^2},$$

where w is the equivalent load in pounds per lineal foot and l is the length of the span in feet. But if one desires to be extremely accurate he should find the centre of gravity of the total live load on the span and move the diagram until the said centre of gravity and the wheel load formerly at mid-span are equidistant from that point, then compute the moment at the wheel and substitute it for M in the formula. In finding the equivalent uniform loads for trusses, if one will figure on cars preceding as well as following the locomotives, and will compute the maximum moments at the quarter points of the spans, then substitute them for M in the proper formula, he will obtain the best averages for the equivalent load curves. Even then he will find that the plotted points do not lie on a regular curve, but that one can be drawn through some and very near to others; and it is this curve that should be used instead of an irregular line joining all the plotted points. As an illustration of this the diagram of equivalent loads for plate-girder spans and Class 50 loading is given in Fig. 10*a*. It will be seen that there are great irregularities for spans between ten and fifteen feet and for those between ninety-five and one hundred and ten feet. These are due mainly to peculiarities in wheel spacing that cause loads to pass on and off the span with a very short motion of train. The author maintains that it is more scientific, more in accordance with common sense, *and, in fact, more accurate,* to take the live loads from the regular curve than it would be to use those found by the exact method of wheel concentrations, because these irregularities are due only to the fact that a typical locomotive was assumed. Surely it is evident to any thoughtful man that the maximum bending moments should increase *regularly* with the span length and not by fits and starts; and that the equivalent load line should lower *regularly* in order to represent *a fair average* equivalent for all locomotives of the heaviest type rather than an exact equivalent for any one assumed locomotive!

The method of applying the equivalent uniform loads in bridge designing is as follows, beginning, as is customary, with the stringers, then passing to the floor-beams and finally to the trusses.

STRINGERS

From Fig. 6*d* find the equivalent live load per lineal foot for a span equal to the panel length, add to same the percentage thereof for impact, as given in Fig. 7*c*, and the assumed weight per foot of the stringers and the floor they support, and divide the sum by two (or by four, if there be four lines of stringers per track), calling the result w; then find the total bending moment at mid-span by substituting in the well-known formula,

$$M = \tfrac{1}{8} w l^2,$$

where l is the panel length in feet, and M is the required moment in foot-pounds.

Should the total end shear be required, it can be found for each stringer by adding together one-half or one-quarter (according to the number of lines of stringers employed) of the end shear given in Fig. 6*c*, the proper

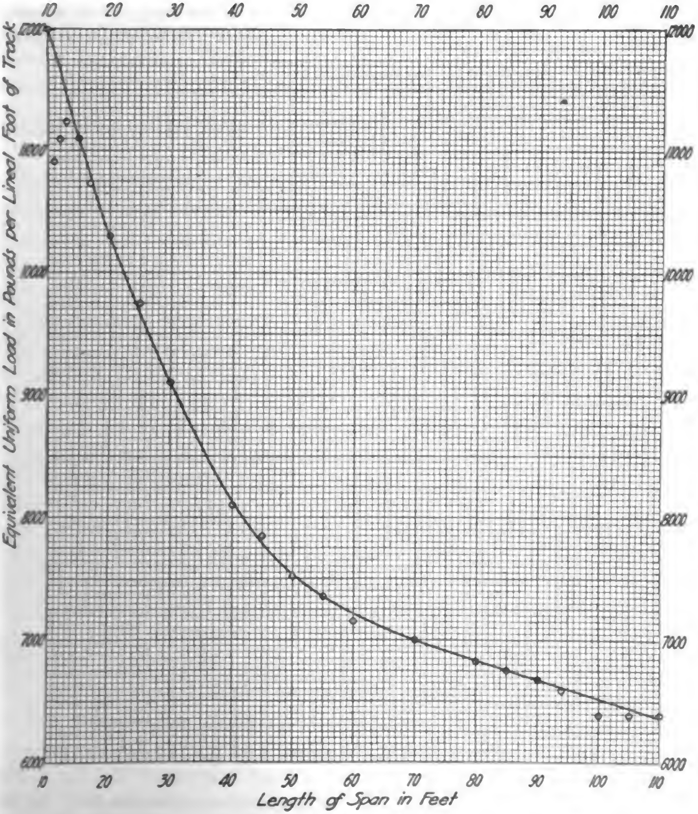


FIG. 10*a*. Equivalent Live Loads for Plate-girder Spans for Class 50.

percentage thereof for impact, and the half weight of one stringer with the floor that it carries.

FLOOR-BEAMS

In proportioning a floor-beam, the important thing to ascertain is the total concentration at the point where two stringers meet. The live-load concentration is to be found by multiplying together the panel length and the equivalent uniform load per lineal foot given in Fig. 6*d for a span equal to twice the panel length*, and dividing the product by two (or by four, if there be four lines of stringers per track). It is unnecessary to describe here how the dead-load concentration at each stringer support is to be found. Nor is it requisite to do more than merely to mention that either the live-load concentration obtained for the floor-beam or a multiple thereof will be the live-load stress in the hanger which supports the beam.

TRUSSES

These can be divided into two kinds, viz., those with equal panels and parallel chords, and those in which the panel lengths are unequal, or the chords are not parallel, or both. In the first case the stresses can be determined most expeditiously by substitution in tabulated formulæ. and in the second by the graphical method.

Case I—Parallel Chords

From Fig. 6*e* find the equivalent uniform live load per lineal foot for the given span length and multiply the same by the panel length, calling the product *L*. For single-track bridges this must be divided by two. All the live-load stresses in main truss-members of single-intersection bridges can be found by substituting this value of *L* in one of the Tables 10*c* to 10*f*, inclusive. Just here it is proper to remark that, strictly speaking, the "Equivalent Uniform-Load Method" is not accurately applicable to trusses of multiple intersection; but the most approved modern practice in bridge engineering does not countenance the building of trusses or girders having more than a single system of cancellation. The "Equivalent Uniform-Load Method" does, however, apply to trusses with divided panels, such as the Petit truss; but as this style of truss nowadays involves almost invariably a polygonal top chord, its treatment herein will come under

Case II—Polygonal Chords

Where trusses have unequal panels or chords not parallel, the first step to take is the finding of all the dead-load stresses by the graphical method, starting from one end of the span and working toward the middle, where the last stress is checked by the method of moments and the correctness of the entire graphical work is thereby proved.

The next step is to find from Fig. 6*e*, as in Case I, the equivalent live-load per lineal foot for the span, and therefrom the value of the panel-truss live load L. Next set a slide-rule for the ratio of dead load per lineal foot and the equivalent live load per lineal foot for the span, and, by referring to the dead-load stresses already found, read off from the rule all of the live-load stresses in chords and inclined end posts.

Next assume that there is an upward reaction at one end of the span equal to 10,000 pounds, or 100,000 pounds (according to the size of the bridge), caused by a load placed at the first panel point from the other end of the span, then find graphically the stress in each web-member from end to end that is caused by this assumed upward reaction, checking the work analytically by computing the chord stress at the first panel point just mentioned. Then calculate the value of the live-load reaction for the maximum stress in each web-member by means of the slide-rule and the following formula and Table 10*b*, in which n is the number of panels in the span, n' is the number of the panel point at the head of the train, counting from the loaded end of the span, and C is the coefficient of $\frac{L}{n}$:

$$\text{Live-load reaction for the head of train at } n' = C \times \frac{L}{n}.$$

TABLE 10*b*

COEFFICIENTS FOR LIVE LOAD REACTIONS

n'	C	n'	C	n'	C	n'	C
1	1	7	28	13	91	19	190
2	3	8	36	14	105	20	210
3	6	9	45	15	120	21	231
4	10	10	55	16	136	22	253
5	15	11	66	17	153	23	276
6	21	12	78	18	171	24	300

Then, still using the slide-rule, find the greatest live-load stress in each web-member by the following equation:

$$\text{Stress required} = \text{Stress from Assumed Reaction} \times \frac{\text{Actual Reaction}}{\text{Assumed Reaction}}.$$

Where the panels are divided as in the Petit truss, and where inclined sub-posts are employed, the *tensile* stress in the *upper* half of each main diagonal thus found will have to be corrected by subtracting therefrom a stress equal to $\frac{L \sec \alpha}{2\,(1 + \tan \alpha \tan \theta)}$, where α is the inclination of the diagonal to the vertical, and θ is the inclination of the chord member in the same panel to the horizontal. With parallel chords this expression evidently becomes $\frac{L}{2} \sec \alpha$. The compressive stress in any main post will have to be corrected by subtracting therefrom a stress equal to $\frac{L}{2}$, for either inclined or parallel chords. No corrections are needed when computing the counter stresses in the web members.

When the panels are divided as in the Petit truss, and inclined subties are used, the *tensile* stress computed by the foregoing method in the *upper* half of each main diagonal will require no correction; and the *tensile* stress in the *lower* half can be found by subtracting from that in the *upper* half a stress equal to $\frac{L}{2}\sec\alpha$. The *compressive* stress found in the *upper* half of each main diagonal requires no correction; and the *compressive* stress in the *lower* half is equal to that in the *upper* half. The *compressive* stress in each main post will require no correction; but the *tensile* stress therein must be corrected by subtracting therefrom a stress equal to $\frac{L}{2}(1 - \tan\alpha\tan\theta)$. With parallel chords this expression evidently becomes $\frac{L}{2}$.

In comparing the equivalent loads for plate-girder spans as given in Fig. 6*d* with those for truss spans as given in Fig. 6*e* for span-lengths common to both, a small discrepancy will be found. This is due to the fact that in plate-girder spans the moment is figured at mid-span while in truss spans it is taken at the quarter points.

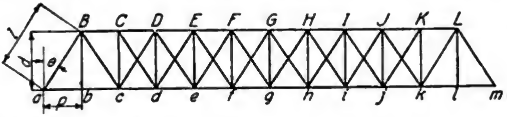
There are still a few engineers and professional writers who talk of the use of the "single-concentration" or the "double-concentration" method of computing stresses in girders and trusses, and it is possible, although not probable, that there are still a few computers who actually employ them, notwithstanding the fact that the author proved in an unanswerable manner as long ago as the early nineties that these methods are not as accurate as the equivalent uniform-load method; and that they involve the expenditure of two or three times as much time and labor. Any one who is curious to read up this bit of ancient history will find in Mr. Harrington's book of "Principal Professional Papers" all of the calculations and arguments advanced at the time the question was under discussion by the engineering profession.

This chapter would not be complete without some reference to influence lines. They provide a very pretty method of computing and indicating stresses, but involve the adoption of wheel loads, which, as previously stated, the author has proved, for many types of structures at least, to necessitate far more labor than do the equivalent uniform loads. It will be sufficient, therefore, to refer the reader for information on this subject to the latest edition of Johnson, Bryan, and Turneaure's "Theory and Practice of Modern Framed Structures," Howe's "Influence Diagrams," and other standard works on bridge stresses.

The following tables of dead-load and live-load stresses for Pratt and Triangular truss spans with parallel chords and of wind-load stresses in chords and lateral systems will be found very useful by the bridge designer:

TABLE 10c

MAXIMUM STRESSES UNDER DEAD AND LIVE LOADS IN PRATT TRUSSES WITH PARALLEL CHORDS



W = Dead Load per panel.
L = Live Load per panel.
Sec $\theta = \dfrac{l}{d}$
Tan $\theta = \dfrac{p}{2d}$
p = Panel length.
d = Depth of truss.
l = Length of diagonal.

For any other truss, letter vertices in manner shown.

The Live and Dead Loads are uniform per foot of span.

All panels are of equal length.

The Dead Load is assumed as concentrated at lower vertices of trusses for through-bridges and at upper vertices of trusses for deck-bridges.

Member		12-panel Truss	11-panel Truss	10-panel Truss	9-panel Truss	8-panel Truss	Multiply by
aB		W5.5 + L5.5	W 5 + L5	W4.5 + L4.5	W 4 + L 4	W3.5 + L3.5	Length of member divided by depth of truss = sec θ.
Bc		" 4.5 + " 55/12	" 4 + " 45/11	" 3.5 + " 36/10	" 3 + " 28/9	" 2.5 + " 21/8	
Cd		" 3.5 + " 45/12	" 3 + " 36/11	" 2.5 + " 28/10	" 2 + " 21/9	" 1.5 + " 15/8	
De		" 2.5 + " 36/12	" 2 + " 28/11	" 1.5 + " 21/10	" 1 + " 15/9	" 0.5 + " 10/8	
Ef		" 1.5 + " 28/12	" 1 + " 21/11	" 0.5 + " 15/10	" 0 + " 10/9	− " 0.5 + " 6/8	
Fg		" 0.5 + " 21/12	" 0 + " 15/11	− " 0.5 + " 10/10	− " 1 + " 6/9	− " 1.5 + " 3/8	
Gh		− " 0.5 + " 15/12	− " 1 + " 10/11	− " 1.5 + " 6/10	− " 2 + " 3/9		
Hi		− " 1.5 + " 10/12	− " 2 + " 6/11				
	abc	W 5.5 + L 5.5	W 5 + L 5	W 4.5 + L 4.5	W 4 + L 4	W3.5 + L3.5	Panel length divided by depth of truss = tan θ.
BC	cd	" 10.0 + " 10.0	" 9 + " 9	" 8.0 + " 8.0	" 7 + " 7	" 6.0 + " 6.0	
CD	de	" 13.5 + " 13.5	" 12 + " 12	" 10.5 + " 10.5	" 9 + " 9	" 7.5 + " 7.5	
DE	ef	" 16.0 + " 16.0	" 14 + " 14	" 12.0 + " 12.0	" 10 + " 10	" 8.0 + " 8.0	
EF	fg	" 17.5 + " 17.5	" 15 + " 15	" 12.5 + " 12.5			
FG		" 18.0 + " 18.0					
Thro	Deck						
	Cc	W4.5 + L 55/12	W 4 + L 45/11	W3.5 + L 36/10	W 3 + L 28/9	W2.5 + L 21/8	Unity.
Cc	Dd	" 3.5 + " 45/12	" 3 + " 36/11	" 2.5 + " 28/10	" 2 + " 21/9	" 1.5 + " 15/8	
Dd	Ee	" 2.5 + " 36/12	" 2 + " 28/11	" 1.5 + " 21/10	" 1 + " 15/9	" 0.5 + " 10/8	
Ee	Ff	" 1.5 + " 28/12	" 1 + " 21/11	" 0.5 + " 15/10	" 0 + " 10/9	− " 0.5 + " 6/8	
Ff	Gg	" 0.5 + " 21/12	" 0 + " 15/11	− " 0.5 + " 10/10			
Gg		− " 0.5 + " 15/12					

Member		7-panel Truss	6-panel Truss	5-panel Truss	4-panel Truss	3-panel Truss	Multiply by
	aB	W 3 + L 3	W2.5 + L2.5	W 2 + L 2.0	W1.5 + L1.5	W 1 + L 1	Length of member divided by depth of truss.
	Bc	" 2 + " 15/7	" 1.5 + " 10/6	" 1 + " 6/5	" 0.5 + " 3/4	" 0 + " 1/3	
	Cd	" 1 + " 10/7	" 0.5 + " 6/6	" 0 + " 3/5	− " 0.5 + " 1/4		
	De	" 0 + " 6/7	− " 0.5 + " 3/6	− " 1 + " 1/5			
	Ef	− " 1 + " 3/7					
	abc	W 3 + L 3	W2.5 + L2.5	W 2 + L 2	W1.5 + L1.5	W 1 + L 1	Panel length div. by depth of truss.
BC	cd	" 5 + " 5	" 4.0 + " 4.0	" 3 + " 3	" 2.0 + " 2.0	" 1 + " 1	
CD	de	" 6 + " 6	" 4.5 + " 4.5				
Thro	Deck						
	Cc	W 2 + L 15/7	W1.5 + L 10/6	W 1 + L 6/5	W0.5 + L 3/4		Unity.
Cc	Dd	" 1 + " 10/7	" 0.5 + " 6/6	" 0 + " 3/5	− " 0.5 + " 1/4		
Dd	Ee	" 0 + " 6/7	− " 0.5 + " 3/6				

TABLE 10*d*

MAXIMUM STRESSES UNDER DEAD AND LIVE LOADS IN THROUGH TRIANGULAR TRUSSES

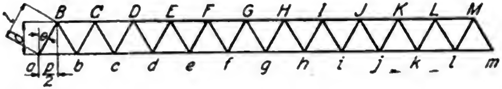
W = Dead Load per panel.
L = Live Load per panel.

$\text{Sec } \theta = \frac{l}{d}$

$\text{Tan } \theta = \frac{p}{2d}$

p = Panel length.
d = Depth of truss.
l = Length of the diagonal.

For any other truss, letter vertices in manner shown.
The Live and Dead Loads are uniform per foot of span.
All panels are of equal length.
The Dead Load is assumed as concentrated at lower vertices of trusses.

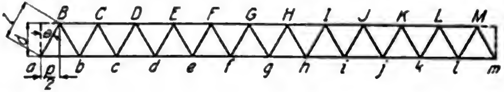


Members		12-Panel Truss	11-Panel Truss	10-Panel Truss	9-Panel Truss	8-Panel Truss	7-Panel Truss	6-Panel Truss	5-Panel Truss	4-Panel Truss	3-Panel Truss	Multiply by
aB	*Bb*	*W*5.5 + *L*5.5	*W* 5 + *L* 5	*W*4.5 + *L* 4.5	*W* 4 + *L* 4	*W*3.5 + *L* 3.5	*W* 3 + *L* 3	*W*2.5 + *L*2.5	*W* 2 + *L* 2	*W*1.5 + *L*1.5	*W* 1 + *L* 1	Length of member divided by depth of truss = sec. θ.
bC	*Cc*	" 4.5 + " 55/12	" 4 + " 45/11	" 3.5 + " 36/10	" 3 + " 28/9	" 2.5 + " 21/8	" 2 + " 15/7	" 1.5 + " 10/6	" 1 + " 6/5	" 0.5 + " 3/4	" 0 + " 1/3	
cD	*Dd*	" 3.5 + " 45/12	" 3 + " 36/11	" 2.5 + " 28/10	" 2 + " 21/9	" 1.5 + " 15/8	" 1 + " 10/7	" 0.5 + " 6/6	" 0 + " 3/5	– " 0.5 + " 1/4		
dE	*Ee*	" 2.5 + " 36/12	" 2 + " 28/11	" 1.5 + " 21/10	" 1 + " 15/9	" 0.5 + " 10/8	" 0 + " 6/7	– " 0.5 + " 3/6	– " 1 + " 1/5			
eF	*Ff*	" 1.5 + " 28/12	" 1 + " 21/11	" 0.5 + " 15/10	" 0 + " 10/9	– " 0.5 + " 6/8	– " 1 + " 3/7					
fG	*Gg*	" 0.5 + " 21/12	" 0 + " 15/11	– " 0.5 + " 10/10	– " 1 + " 6/9	– " 1.5 + " 3/8						
gH	*Hh*	– " 0.5 + " 15/12	– " 1 + " 10/11	– " 1.5 + " 6/10	– " 2 + " 3/9							
hI	*Ii*	– " 1.5 + " 10/12	– " 2 + " 6/11									
ab		*W* 5.5 + *L* 5.5	*W* 5 + *L* 5	*W* 4.5 + *L* 4.5	*W* 4 + *L* 4	*W* 3.5 + *L* 3.5	*W* 3 + *L* 3	*W*2.5 + *L*2.5	*W* 2 + *L* 2	*W*1.5 + *L*1.5	*W* 1 + *L* 1	One-half panel length divided by depth of truss = tan θ.
bc		" 15.5 + " 15.5	" 14 + " 14	" 12.5 + " 12.5	" 11 + " 11	" 9.5 + " 9.5	" 8 + " 8	" 6.5 + " 6.5	" 5 + " 5	" 3.5 + " 3.5	" 2 + " 2	
cd		" 23.5 + " 23.5	" 21 + " 21	" 18.5 + " 18.5	" 16 + " 16	" 13.5 + " 13.5	" 11 + " 11	" 8.5 + " 8.5	" 6 + " 6			
de		" 29.5 + " 29.5	" 26 + " 26	" 22.5 + " 22.5	" 19 + " 19	" 15.5 + " 15.5	" 12 + " 12					
ef		" 33.5 + " 33.5	" 29 + " 29	" 24.5 + " 24.5	" 20 + " 20							
g		" 35.5 + " 35.5	" 30 + " 30									
BC		*W* 11 + *L* 11	*W* 10 + *L* 10	*W* 9 + *L* 9	*W* 8 + *L* 8	*W* 7 + *L* 7	*W* 6 + *L* 6	*W* 5 + *L* 5	*W* 4 + *L* 4	*W* 3 + *L* 3	*W* 2 + *L* 2	
CD		" 20 + " 20	" 18 + " 18	" 16 + " 16	" 14 + " 14	" 12 + " 12	" 10 + " 10	" 8 + " 8	" 6 + " 6	" 4 + " 4		
DE		" 27 + " 27	" 24 + " 24	" 21 + " 21	" 18 + " 18	" 15 + " 15	" 12 + " 12	" 9 + " 9				
EF		" 32 + " 32	" 28 + " 28	" 24 + " 24	" 20 + " 20	" 16 + " 16						
FG		" 35 + " 35	" 30 + " 30	" 25 + " 25								
GH		" 36 + " 36										

TABLE 10e

MAXIMUM STRESSES UNDER DEAD AND LIVE LOADS IN DECK TRIANGULAR TRUSSES

W = Dead Load per panel.
L = Live Load per panel.
$\sec\theta = \frac{l}{d}$
$\tan\theta = \frac{p}{2d}$
p = Panel length.
d = Depth of truss.
l = Length of the diagonal.

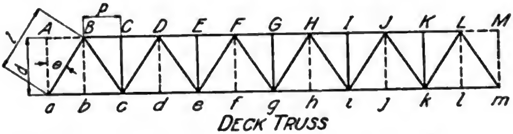


For any other truss, letter vertices in manner shown.
The Live and Dead Loads are **uniform per foot of span.**
All panels are of equal length.
The Dead Load is assumed as concentrated at upper vertices of trusses.

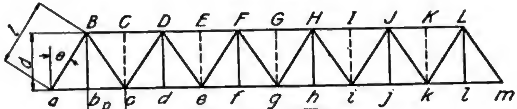
Members		12-Panel Truss	11-Panel Truss	10-Panel Truss	9-Panel Truss	8-Panel Truss	7-Panel Truss	6-Panel Truss	5-Panel Truss	4-Panel Truss	3-Panel Truss	Multiply by
aB		W 6 + L 6	W5.5 + L5.5	W 5 + L 5	W4.5 + L4.5	W 4 + L 4	W3.5 + L3.5	W 3 + L 3	W2.5 + L2.5	W 2 + L 2	W1.5 + L1.5	Length of member divided by depth of truss = sec. θ.
Bb	bC	" 5 + " 121/24	" 4.5 + " 100/22	" 4 + " 81/20	" 3.5 + " 64/18	" 3 + " 49/16	" 2.5 + " 36/14	" 2 + " 25/12	" 1.5 + " 16/10	" 1 + " 9/8	" 0.5 + " 4/6	
Cc	cD	" 4 + " 100/24	" 3.5 + " 81/22	" 3 + " 64/20	" 2.5 + " 49/18	" 2 + " 36/16	" 1.5 + " 25/14	" 1 + " 16/12	" 0.5 + " 9/10	" 0 + " 4/8	− " 0.5 + " 1/6	
Dd	dE	" 3 + " 81/24	" 2.5 + " 64/22	" 2 + " 49/20	" 1.5 + " 36/18	" 1 + " 25/16	" 0.5 + " 16/14	" 0 + " 9/12	− " 0.5 + " 4/10	− " 1 + " 1/8		
Ee	eF	" 2 + " 64/24	" 1.5 + " 49/22	" 1 + " 36/20	" 0.5 + " 25/18	" 0 + " 16/16	− " 0.5 + " 9/14	− " 1 + " 4/12				
Ff	fG	" 1 + " 49/24	" 0.5 + " 36/22	" 0 + " 25/20	− " 0.5 + " 16/18	− " 1 + " 9/16	− " 1.5 + " 4/14					
Gg	gH	" 0 + " 36/24	− " 0.5 + " 25/22	− " 1 + " 16/20	− " 1.5 + " 9/18							
Hh	hI	− " 1 + " 25/24	− " 1.5 + " 16/22									
Ii	iJ	− " 2 + " 16/24										
ab		W 6 + L 6	W 5.5 + L 5.5	W 5 + L 5	W 4.5 + L 4.5	W 4 + L 4	W 3.5 + L 3.5	W 3 + L 3	W2.5 + L2.5	W 2 + L 2	W1.5 + L1.5	One-half panel length divided by depth of truss = tan θ.
bc		" 16 + " 16	" 14.5 + " 14.5	" 13 + " 13	" 11.5 + " 11.5	" 10 + " 10	" 8.5 + " 8.5	" 7 + " 7	" 5.5 + " 5.5	" 4 + " 4	" 2.5 + " 2.5	
cd		" 24 + " 24	" 21.5 + " 21.5	" 19 + " 19	" 16.5 + " 16.5	" 14 + " 14	" 11.5 + " 11.5	" 9 + " 9	" 6.5 + " 6.5			
de		" 30 + " 30	" 26.5 + " 26.5	" 23 + " 23	" 19.5 + " 19.5	" 16 + " 16	" 12.5 + " 12.5					
ef		" 34 + " 34	" 29.5 + " 29.5	" 25 + " 25	" 20.5 + " 20.5							
fg		" 36 + " 36	" 30.5 + " 30.5									
BC		W 11 + L 11	W 10 + L 10	W 9 + L 9	W 8 + L 8	W 7 + L 7	W 6 + L 6	W 5 + L 5	W 4 + L 4	W 3 + L 3	W 2 + L 2	
CD		" 20 + " 20	" 18 + " 18	" 16 + " 16	" 14 + " 14	" 12 + " 12	" 10 + " 10	" 8 + " 8	" 6 + " 6	" 4 + " 4		
DE		" 27 + " 27	" 24 + " 24	" 21 + " 21	" 18 + " 18	" 15 + " 15	" 12 + " 12	" 9 + " 9				
EF		" 32 + " 32	" 28 + " 28	" 24 + " 24	" 20 + " 20	" 16 + " 16						
FG		" 35 + " 35	" 30 + " 30	" 25 + " 25								
GH		" 36 + " 36										

TABLE 10f

MAXIMUM STRESSES UNDER DEAD AND LIVE LOADS IN DECK AND THROUGH TRIANGULAR TRUSSES WITH VERTICALS



DECK TRUSS



THROUGH TRUSS

W = Dead Load per Panel.
L = Live Load per Panel.

$$\text{Sec } \theta = \frac{l}{d}$$

$$\text{Tan } \theta = \frac{p}{d}$$

p = Panel length.
d = Depth of truss.
l = Length of diagonal.

For any other truss, letter vertices in manner shown.

The Live and Dead Loads are uniform per foot of span.

All panels are of equal length.

The Dead Load is assumed as concentrated at lower vertices of trusses for through-bridges and at upper vertices of trusses for deck-bridges.

Members	12-Panel Truss	10-Panel Truss	8-Panel Truss	6-Panel Truss	4-Panel Truss	Multiplied by
aB	$W5.5+L5.5$	$W4.5+L4.5$	$W3.5+L3.5$	$W2.5+L2.5$	$W1.5+L1.5$	Length of the member divided by the depth of truss = sec. θ.
Bc	" 4.5 + " 55/12	" 3.5 + " 36/10	" 2.5 + " 21/8	" 1.5 + " 10/6	" 0.5 + " 3/4	
cD	" 3.5 + " 45/12	" 2.5 + " 28/10	" 1.5 + " 15/8	" 0.5 + " 6/6	− " 0.5 + " 1/4	
De	" 2.5 + " 36/12	" 1.5 + " 21/10	" 0.5 + " 10/8	− " 0.5 + " 3/6		
eF	" 1.5 + " 28/12	" 0.5 + " 15/10	− " 0.5 + " 6/8			
Fg	+ " 0.5 + " 21/12	− " 0.5 + " 10/10	− " 1.5 + " 3/8			
gH	− " 0.5 + " 15/12	− " 1.5 + " 6/10				
Hi	− " 1.5 + " 10/12					
Full Verticals....	$W+L$	$W+L$	$W+L$	$W+L$	$W+L$	Unity
Dotted Verticals....	Zero	Zero	Zero	Zero	Zero	
abc	$W\ 5.5+L\ 5.5$	$W\ 4.5+L\ 4.5$	$W3.5+L3.5$	$W2.5+L2.5$	$W1.5+L1.5$	Panel length divided by the depth of truss = tan θ.
cde	" 13.5 + " 13.5	" 10.5 + " 10.5	" 7.5 + " 7.5	" 4.5 + " 4.5		
efg	" 17.5 + " 17.5	" 12.5 + " 12.5				
BCD	$W\ 10+L\ 10$	$W\ 8+L\ 8$	$W\ 6+L\ 6$	$W\ 4+L\ 4$	$W\ 2+L\ 2$	
DEF	" 16 + " 16	" 12 + " 12	" 8 + " 8	" 6 + " 6		
FGH	" 18 + " 18					

TABLE 10*g*

Coefficients of $W \tan \theta$ for Both Compression and Tension Stresses in Bottom Chords of Through-Bridges and Top Chords of Deck-Bridges, Due to Wind Loads Applied to Said Chords, when the Lateral System is of Double Cancellation

Number of Panels in Span	Number of Panel from End of Span												
	1	2	3	4	5	6	7	8	9	10	11	12	13
4	¾	1¾											
5	1	2½	3										
6	1¼	3¼	4¼										
7	1½	4	5½	6									
8	1¾	4¾	6¾	7¾									
9	2	5½	8	9½	10								
10	2¼	6¼	9¼	11¼	12¼								
11	2½	7	10½	13	14½	15							
12	2¾	7¾	11¾	14¾	16¾	17¾							
13	3	8½	13	16½	19	20½	21						
14	3¼	9¼	14¼	18¼	21¼	23¼	24¼						
15	3½	10	15½	20	23½	26	27½	28					
16	3¾	10¾	16¾	21¾	25¾	28¾	30¾	31¾					
17	4	11½	18	23½	28	31½	34	35½	36				
18	4¼	12¼	19¼	25¼	30¼	34¼	37¼	39¼	40¼				
19	4½	13	20½	27	32½	37	40½	43	44½	45			
20	4¾	13¾	21¾	28¾	34¾	39¾	43¾	46¾	48¾	49¾			
21	5	14½	23	30½	37	42½	47	50½	53	54½	55		
22	5¼	15¼	24¼	32¼	39¼	45¼	50¼	54¼	57¼	59¼	60¼		
23	5½	16	25½	34	41½	48	53½	58	61½	64	65½	66	
24	5¾	16¾	26¾	35¾	43¾	50¾	56¾	61¾	65¾	68¾	70¾	71¾	
25	6	17½	28	37½	46	53½	60	65½	70	73½	76	77½	78
26	6¼	18¼	29¼	39¼	48¼	56¼	63¼	69¼	74¼	78¼	81¼	83¼	84¼

TABLE 10*h*

Coefficients of $\frac{W \sec \theta}{n}$ (Where n = No. of Panels in Span) for Wind-Load Stresses in the Diagonals of Lateral Systems of Single Cancellation. These Coefficients Apply to Lateral Systems Composed of Intersecting Diagonal Rods or of Single Diagonal Struts

No. of Panels in Span	Number of Panel from End of Span												
	1	2	3	4	5	6	7	8	9	10	11	12	13
4	6	3											
5	10	6	3										
6	15	10	6										
7	21	15	10	6									
8	28	21	15	10									
9	36	28	21	15	10								
10	45	36	28	21	15								
11	55	45	36	28	21	15							
12	66	55	45	36	28	21							
13	78	66	55	45	36	28	21						
14	91	78	66	55	45	36	28						
15	105	91	78	66	55	45	36	28					
16	120	105	91	78	66	55	45	36					
17	136	120	105	91	78	66	55	45	36				
18	153	136	120	105	91	78	66	55	45				
19	171	153	136	120	105	91	78	66	55	45			
20	190	171	153	136	120	105	91	78	66	55			
21	210	190	171	153	136	120	105	91	78	66	55		
22	231	210	190	171	153	136	120	105	91	78	66		
23	253	231	210	190	171	153	136	120	105	91	78	66	
24	276	253	231	210	190	171	153	136	120	105	91	78	
25	300	276	253	231	210	190	171	153	136	120	105	91	78
26	325	300	276	253	231	210	190	171	153	136	120	105	91

NOTE.—For the stresses in diagonals of lateral systems of double cancellation, *i.e.*, those systems in which the diagonals are composed of intersecting struts, divide the coefficients in the above table by two.

TABLE 10*i*

COEFFICIENTS OF $W \tan \theta$ FOR COMPRESSION-STRESSES IN WINDWARD BOTTOM CHORDS OF THROUGH-BRIDGES, AND WINDWARD TOP CHORDS OF DECK-BRIDGES, DUE TO WIND LOADS APPLIED DIRECTLY TO SAID CHORDS, WHEN THE LATERAL SYSTEM IS OF SINGLE CANCELLATION. THE TENSILE STRESSES IN LEEWARD CHORDS ARE NUMERICALLY EQUAL TO THE COMPRESSION STRESSES GIVEN IN THE TABLE FOR ONE PANEL NEARER END OF SPAN

No. of Panels in Span	Number of Panel from End of Span												
	1	2	3	4	5	6	7	8	9	10	11	12	13
4	1½	2											
5	2	3	3										
6	2½	4	4½										
7	3	5	6	6									
8	3½	6	7½	8									
9	4	7	9	10	10								
10	4½	8	10½	12	12½								
11	5	9	12	14	15	15							
12	5½	10	13½	16	17½	18							
13	6	11	15	18	20	21	21						
14	6½	12	16½	20	22½	24	24½						
15	7	13	18	22	25	27	28	28					
16	7½	14	19½	24	27½	30	31½	32					
17	8	15	21	26	30	33	35	36	36				
18	8½	16	22½	28	32½	36	38½	40	40½				
19	9	17	24	30	35	39	42	44	45	45			
20	9½	18	25½	32	37½	42	45½	48	49½	50			
21	10	19	27	34	40	45	49	52	54	55	55		
22	10½	20	28½	36	42½	48	52½	56	58½	60	60½		
23	11	21	30	38	45	51	56	60	63	65	66	66	
24	11½	22	31½	40	47½	54	59½	64	67½	70	71½	72	
25	12	23	33	42	50	57	63	68	72	75	77	78	78
26	12½	24	34½	44	52½	60	66½	72	76½	80	82½	84	84½

CHAPTER XI

SECONDARY STRESSES, TEMPERATURE STRESSES, AND INDETERMINATE STRESSES

SECONDARY STRESSES

By the term "secondary stresses" is meant those induced or indirect stresses which occur as a necessary result of the deformations caused by the primary actions of a structure when loaded. The principal stresses in the various parts are computed under the assumption that all members are connected at the joints by frictionless pins, on which they can turn freely. This assumption, of course, is never realized. Most of the connections in modern steel bridges are riveted ones, the gain in stiffness and rigidity attained by so making them being highly desirable, although their use results in certain distortions and bending stresses of more or less magnitude; and even where pins are employed, they are far from being frictionless.

The quantitative analysis of these secondary stresses is a comparatively recent development in American practice, although the general methods therefor were developed some twenty or thirty years ago, principally by German writers. Within the past few years several treatments of the subject have appeared in this country. The book which was used most largely in the earlier calculations made in the author's office was C. R. Grimm's work on "Secondary Stresses in Bridge Trusses." It covers quite satisfactorily the various methods of analysis. The author takes great satisfaction in the thought that that valuable work, as stated in its preface, owes its existence to him. For several years he searched faithfully for a German-American bridge engineer who had had the requisite experience and could spare the time for writing a condensed, practical treatise upon the subject of secondary stresses in bridge trusses. He found only a few such engineers who possessed the necessary knowledge; and of these Mr. Grimm was the only one who could afford the time for writing the book. The manner in which he did it reflects great credit upon him and indirectly a little, possibly, upon the "instigator." The treatment given in Part II of "Modern Framed Structures," which appeared later than Grimm's book, is, perhaps, better adapted to the direct use of the designer, as it devotes considerable space to showing how the amount of labor required can be reduced materially by a systematic arrangement of the calculations. Mr. F. C. Kunz, in an article in *Engineering News*, Vol. 66,

p. 397, gives a very good presentation of the subject, using a method of analysis first proposed by Mohr. (A few typographical errors in this article are noted on p. 515 of the same volume.) A particularly complete discussion, from the practical standpoint, is to be found in a committee report presented to the American Railway Engineering Association in 1914, and printed in Vol. 15 of its *Proceedings*. It covers the general features of the subject and explains in detail the method of calculation of stresses in the plane of the trusses, presenting the complete computations for one truss. It also compares the secondary stresses in several different types of trussing, and furthermore gives the results of secondary stress measurements on several bridges. The method of analysis used in this report is quite similar to that given in "Modern Framed Structures."

This report divides the secondary stresses into five classes, as follows:

"1. Bending stresses in the plane of the main truss due to the rigidity of joints, eccentricity of joints, and weight of members.

"2. Bending stresses in members of a transverse frame due to the deflection of floor-beams and to primary stresses in posts.

"3. Stresses in a horizontal plane due to longitudinal deformation of chords, especially the stresses in floor-beams and in their connections.

"4. Variation of axial stress in different elements of a member.

"5. Stresses due to vibration of individual members."

The stresses under Nos. 1, 2, and 3 can be analyzed more or less completely, but those under Nos. 4 and 5 can not be as accurately determined. The last two will not be considered further in this chapter.

Bending Stresses in the Plane of the Main Truss Due to the Rigidity of the Joints

These stresses are the ones which are generally referred to when the term "secondary stresses" is employed. In the earlier days of American bridgework, nearly all important structures were pin-connected, and they were assumed to be substantially free from stresses of this nature. With the increasing adoption of riveted connections and stiff members, and with the appreciation of the fact that the use of pins does not necessarily do away with these bending effects entirely, the need for careful investigation and analysis along this line has become apparent. A great deal has already been accomplished; but there is still left a wide field for research and experiments.

No attempt will be made in this book to treat fully the various methods of analysis that have been proposed, the reader being referred to the authorities above mentioned. However, the assumptions and formulæ of the ordinary method of analysis will be briefly discussed, in order that the underlying principles of the subject may be clearly understood.

If a truss, the members of which are connected at the panel points by frictionless pins, be loaded in any manner, the various members will change in length slightly, the various panel-points will deflect, and the angles made with each other by the various members meeting at each point will alter. The members will remain straight between panel-points, however, as they can rotate freely on the pins. If now a similar truss having rigid joints be considered, the changes in lengths on the members and the deflections of the panel-points will be substantially as before; but the angles between the various members meeting at a panel-point will be forced to remain unchanged. As a result, each joint will rotate as a whole into some such position that equilibrium throughout the truss will be secured, and each member will thereby be bent more or less. Let us suppose that we have under consideration a member *nm*, joining any panel-point *n* with any adjacent panel-point *m*. It is proved in the texts before quoted that the moment in the said member at the point *n* is given with sufficient accuracy by the formula,

$$M_{nm} = \frac{4EI}{l}\left(T_{nm} + \frac{1}{2}T_{mn}\right) \qquad \text{[Eq. 1]}$$

in which M_{nm} = moment in member *nm* at point *n*, positive when it tends to rotate the joint in a clockwise direction,
E = coefficient of elasticity of the material,
I = gross moment of inertia of member *nm*,
l = length of member *nm*,
T_{nm} = angle in radians made by the end tangent of the member *nm* at the point *n* with the straight line joining the panel-points *n* and *m*, to be taken as positive when the said end tangent has rotated in a counter-clockwise direction from the said line,
and T_{mn} = similar angle at *m*.

This equation, and the self-evident requirement

$$\sum M_{nm} = 0 \qquad \text{[Eq. 2]}$$

for each panel-point, are together sufficient for the calculation of the values of the angles T throughout the truss. After these values have been found, the fibre stresses in the various members can be computed by the equation

$$f_{nm} = \frac{M_{mn}\,c}{I} = \frac{4Ec}{l}\left(T_{mn} + \frac{1}{2}T_{mn}\right), \qquad \text{[Eq. 3]}$$

where f_{nm} = extreme fibre stress from bending in member *nm* at the point *n*,
and c = distance of the said fibre from the neutral axis.

Equations 1 and 3 are a trifle unwieldy to use, as the quantities outside of the parentheses are very large, while those within them are very small fractions. However, they can readily be transformed into a more convenient form. It will be found that for a truss loaded with dead load over the entire span and full live and impact loads over all or part of the span the values of the T's will generally be a few ten-thousandths of a radian, usually ranging from 0.0000 to 0.0010, and in exceptional cases to 0.0020, or even more. If, therefore, we call E 30,000,000 and express T in ten-thousandths of a radian, Equation 1 becomes

$$M_{nm} = 12{,}000 \frac{I}{l} \left(T_{nm} + \frac{1}{2} T_{mn} \right), \qquad \text{[Eq. 4]}$$

and Equation 3 becomes

$$f_{mn} = 12{,}000 \frac{c}{l} \left(T_{nm} + \frac{1}{2} T_{mn} \right). \qquad \text{[Eq. 5]}$$

Also, it is evident that instead of Equation 2 we can use the formula

$$\sum \frac{I}{l} \left(T_{nm} + \frac{1}{2} T_{mn} \right) = 0, \qquad \text{[Eq. 6]}$$

expressing the T's in ten-thousandths of a radian in this case also. The use of the equations in the above forms will save a considerable amount of time. The form of Equation 5 is especially convenient, since it will enable one to judge directly what values of T will indicate the presence of high secondary stresses in any particular member.

While the theoretical basis for the calculation of secondary stresses is seen to be quite simple, unfortunately the numerical work is rather lengthy. The labor involved, however, can be much diminished by a systematic arrangement of the calculations, as illustrated in Part II of "Modern Framed Structures," and in the Committee Report of the Am. Ry. Eng. Assn. The analysis of an ordinary truss of short span for a single position of loads will take several hours; and the complete analysis by the method of joint loads will generally require about two days. For a large truss, especially one with subdivided panels, a much longer time will be needed; consequently the desirability of having some shorter method which will give approximately correct results is quite apparent.

In the following pages there will be presented a shorter method of analysis, based on the formulæ given above. By this method it will be possible to obtain in a short time a general idea as to the magnitude of the secondary stresses in the various members of a truss, while by carrying the computations further any desired degree of accuracy may be obtained. The method is by no means an original one with the author, as certain features of it have been used more or less by various investigators. At the end of the chapter (pages 218 to 226, inclusive) will be found complete calculations, worked out by this method, for the secondary stresses in a single-track-railway, through, riveted truss of 296′ span (the one shown in Fig. 11*g*), designed for Class 60 loading; and the reader would do well to keep these calculations before him in reading the following explanation.

Before beginning the calculation of the secondary stresses, it is essential that the general dimensions of the structure and the make-up of the members be already known. There are first determined for each member the length l, the gross area A, the gross moment of inertia I about an axis at right angles to the plane of the truss, and the ratios $\frac{I}{l}$ and 12,000 $\frac{c}{l}$, it being usually possible to take the values of l, A, and I for some or all of the members from the design calculations made previously. Next there are computed in each member, for the assumed condition of loading, the primary stress S and the elongation Δl, using the gross area A in figuring the latter quantity. A Williot diagram is then drawn in the usual manner, assuming any convenient member to remain unchanged in direction. It will make no difference whether the member selected actually rotates or not, since the diagram will be used only to determine the differences between the angles through which the various members turn.

The angle θ through which each member has turned with respect to the member which is considered to remain fixed is figured from the Williot diagram, as explained in the discussion thereof in Chapter XII. Each panel-point—*i.e.*, the end tangents of all members meeting at each point—is then assumed to rotate through some angle β, and the angle $\beta - \theta$ is computed for each member meeting there, the resulting angle being called φ. The angles θ and β are to be considered positive when rotation has been in a counter-clockwise direction. The value of β for any panel-point is best taken as the average of the θ's of the two chord members meeting there, so that the φ's for these two members will be equal. It is desirable to have these φ's as nearly as possible equal to the corresponding T's, which are still unknown, in order to reduce the work of computing the said T's; and the above method attains this result in the best manner that any general rule can. After a designer becomes familiar with the method, he can frequently make somewhat better assumptions in particular cases.

These angles φ are to be considered as the first trial values of the angles T, and Equation 6 is then applied to each panel-point in turn. It will be found that for most points the resulting summations will not be zero. The next step is to adjust the trial values of the T's so as to obtain values more nearly correct. The amount that the said trial values at any panel-point are in error could be determined by dividing the value of the summation for the said point by the sum of the $\frac{I}{l}$'s of the members meeting there, if it were not for the fact that the angles at the other ends of each of these members are also likely to be more or less in error. However, the corrections found in this manner will be approximately right; and the adjustment should be made on that basis. It will be best to correct first those points which evidently need the largest adjustments, and take into account the effects of these changes when making the adjustments at the other points. While the corrections can be made in any order, and correct results still be reached, much time can be saved by carrying through the work in the proper manner. In order to illustrate the method to be followed, the calculations made in the adjustment of the values of the T's for the truss given at the end of the chapter will now be explained in detail. See Fig. 11*i* and Table 11*f*.

As soon as the φ's had been computed in the manner above explained, a small scale diagram of the truss was laid out (Fig. 11*i*), the $\frac{I}{l}$'s of the various members written on them, and the summations of the $\frac{I}{l}$'s of all of the members meeting at any one panel-point written just above or below the said panel-point. Next the values of the angles φ (which will henceforth be called the first trial values of the angles T) were written on the various members, in such positions that it would be possible to write two or three more trial values above or below them, the said angles being expressed in ten-thousandths of a radian. The quantity $\left(T_{nm} + \frac{1}{2}\, T_{mn}\right)$ was then calculated for each end of each member; and the results for all the members meeting at any one panel-point were written down one below the other in Table 11*f*. The figure for the chord member to the left was put down first, then that for the chord member to the right, then that for the diagonal, and lastly that for the post. Each of the quantities $\left(T_{nm} + \frac{1}{2}\, T_{mn}\right)$ was then multiplied by the proper value of $\frac{I}{l}$; and the summations of the products $\frac{I}{l}\left(T_{nm} + \frac{1}{2}\, T_{mn}\right)$ were formed for the various panel-points. The work of adjustment was then begun. At point 4, which evidently needed much more adjustment than any other point, the values of the T's were all increased (algebraically) by $\frac{102}{51.4} = 2.0$ ten-thousandths of a radian. The effects of this adjustment on the quantities $\frac{I}{l}\left(T_{nm} + \frac{1}{2}\, T_{mn}\right)$ at points 2, 3, 5, and 6 were then figured and put down just to the right of the quantity changed, giving + 16 for member 2–4 at 2, + 8 for member 3–4 at 3, + 4 for member 4–5 at 5, and + 23 for member 4–6 at 6. Point 8 was next treated, because it needed considerable adjustment, and also because it was evident that this adjusted value should be pretty nearly correct. This last statement is true for the reasons that neither 6 nor 10 needed much adjustment, and that the adjustments at points 7 and 9 would not affect materially the conditions at 8, as the values of the $\frac{I}{l}$'s of 7–8 and 8–9 were small. The correction required at 8 was $\frac{42}{56.8} = +0.7$, giving a change of + 8 in 6–8 at 6, + 1 in 7–8 at 7, + 1 in 8–9 at 9, and + 9 in 8–10 at 10. Points 1, 3, 5, 7, and 9 all needed considerable adjustment. It was decided to correct 3 first, as it was more in error than any of the others. The adjustment made was $\frac{65}{55.7} = +1.2$, giving a change of + 12 in 1–3 at 1, 0 in 2–3 at 2, and + 16 in 3–5 at 5. (The change in 3–4 at 4 was not figured, as the first adjustment at point 4 had already been made.) The totals of the summations at points 5, 7, and 9 then stood −31, −46, and −46, hence it was somewhat uncertain as to which it would be best to adjust first. Point 7 was selected, as it would tend to give better results at both 5 and 9, leaving point 7 the only one likely to be much in error after completing the adjustment. The change at 7 was $\frac{46}{68.3} = +0.7$, and the resulting changes were + 11 in 5–7 at 5, + 11 in 7–9 at 9, and + 1 in 6–7 at 6. Point 5 was then adjusted by $\frac{20}{65} = +0.3$, giving a change of + 1 in 5–6 at 6. The change required at point 9 was $\frac{35}{70} = +0.5$, causing changes of + 8 in 9–11 at 11, and + 1 in 9–10 at 10. Point 11 was next adjusted. This point was peculiar, in that any change in the value of the T of member 11–13 at 11 was accompanied by an equal change in the same member at 13, due to the fact that the truss and its loading were both symmetrical about 14. Since $T_{mn} = -T_{nm}$, the expression $\frac{I}{l}\left(T_{nm} + \frac{1}{2}T_{mn}\right)$ for this member becomes $\frac{1}{2}\frac{I}{l}T_{nm}$. The $\Sigma\frac{I}{l}$ for point 11 should, therefore, be reduced by one-half of the $\frac{I}{l}$ of 11–13, before using it as a divisor, giving 51.3. The change required at 11 was, therefore, $\frac{17}{51.3} = +0.3$, while that at point 13 was −0.3. Point 10 was next considered, but no change was made there, as the total summation was only −3. Point 6 was then taken up, the required adjustment being $\frac{13}{54.8} = -0.2$. Point 1 was then adjusted, a correction of $\frac{40}{36.5} = -1.1$ being required, giving a change of −9 in 1–2 at 2. The adjustment at Point 2 was then made, the necessary change being $\frac{22}{33.1} = -0.7$. Point 14 required no adjustment, as its true position was evident from the fact that both the truss and its loading were symmetrical about the said point.

The second trial values of the angles T found as above should be about correct. In order to test this fact, as well as to give a check on the work, the summations of the quantities $\frac{I}{l}\left(T_{nm}+\frac{1}{2}T_{mn}\right)$ should again be made up. In the problem above explained, there were summations of -6 at point 1, $+14$ at Point 7 and $+4$ at Point 9, the other summations being 2 or less. Evidently an adjustment of $+0.2$ was required at 1 and -0.2 at 7. The third trial values of the T's were then worked out. Point 1 was first adjusted by $+0.2$; and the resulting changes at Points 2 and 3 were figured. Point 7 was next corrected by -0.2; and the resulting changes at points 5, 9, 6, and 8 were computed, those at 6 and 8 being less than 1. The new summation at 9 was $+1$, indicating that no change was needed there; and that at 5 was -2, for which no correction was required. All other third trial values of the T's were the same as the second trial values. The summations of the quantities $\frac{I}{l}\left(T_{nm}+\frac{1}{2}T_{mn}\right)$ were then worked out from these third trial values. These summations indicated that, in this problem at least, no further changes were necessary.

In general, it will be found unnecessary to carry the work further than the finding of the third trial values. However, if for any reason they should not be near enough to the correct values, the process can evidently be continued until the results are satisfactory.

The method above outlined will be found less laborious than the analytical method usually employed, even if it be thought necessary to carry the work through clear to the check of the summations of the third trial values of the T's, as was done for the sake of illustration in the example above explained. However, the method has the further advantage that it is possible to tell at any stage of the work the probable errors in the trial values of the T's which have been obtained, and thus to decide in an intelligent manner whether it is worth while to carry the work any further. Thus, it is known that even the first trial values of the T's will usually be within one or two ten-thousandths of a radian of the correct values, and by Equation 5 the probable error in the fibre stresses in any member due to the said differences can be determined. This error for ordinary trusses will not exceed 1,000 pounds per square inch. For the truss above discussed the maximum error is 1,000 pounds per square inch; and by comparing the fibre stresses computed from these first trial values with those obtained from the third trial values, it will be noted that the general agreement is close, and that for practical purposes the approximate values would suit very well. It is to be remembered that the important point in a secondary stress investigation is not the determination of the exact values of the said stresses (for it is not to be expected that the figures given by the computations will agree very closely with those in the actual structure), but to determine whether the said stresses are small and unimportant—ranging, say, between zero and 3,000 pounds per square inch—or whether they are large and undesirable. As the ratios $12{,}000\frac{c}{l}$ have already been computed, it takes but a few moments to determine the approximate amounts of the stresses in the various members, before deciding whether it is worth while to proceed further.

In general, however, it will be best at least to check the summations at the various panel points, using the first trial values. As soon as these summations have been obtained, the probable errors in the first trial values of the T's at any panel point can be told pretty closely by dividing the said summation by the sum of the $\frac{I}{l}$'s at the point. If these probable errors be figured in this manner for the truss illustrated in the example at the end of the chapter, and if the results be compared with the differences of the first and third trial values, we find the following results:

Point	Probable Error	Difference of First and Third Trial Values
1	+0.8	+0.9
2	+0.5	+0.7
3	−1.3	−1.2
4	−2.0	−2.0
5	−0.8	−0.3
6	−0.4	+0.2
7	−0.7	−0.5
8	−0.7	−0.7
9	−0.7	−0.5
10	−0.2	0.0
11	−0.4	−0.3

It is noted that the greatest discrepancy is 0.6, there being but one other exceeding 0.2. Evidently if the first trial values of the T's were corrected in the approximate manner just described, the maximum error in any quantity $\left(T_{nm} + \frac{1}{2}T_{mn}\right)$ would be six ten-thousandths of a radian. As the maximum value of $12{,}000\frac{c}{l}$ is a little over 500, the maximum error in any of the fibre stresses would be about 300 pounds per square inch. Evidently we can obtain results sufficiently accurate in many cases by making the corrections in this way. However, as it does not take much time to figure the second trial values in the manner previously outlined, it will usually be worth while to do so. These second trial values will serve for practically any case. In the truss above discussed the maximum error in the second trial values of the T's is 0.2, and in the resulting fibre stresses, 100 pounds per square inch. As a check it will be best to calculate also the summations for the second trial values. From these summations the errors in the second trial values can be told at once, and the question as to the advisability of making any further adjustments can be decided. It may occasionally be necessary to figure the third trial values; and once in a while it may be thought best to figure also the resulting summations.

There is a particular case which might be mentioned in which the method just outlined could be used to advantage. It may occasionally be desirable to determine the probable values in some proposed truss before its design has been made. In this case the truss dimensions and the unit stresses in the various members should be determined, and the first trial values of the T's computed. Now by assuming the values of c for the various members, the approximate values of the secondary stresses can be figured. If results a trifle more accurate are desired, approximate values of the I's can be assumed for the various members, and rough second trial values of the T's computed.

The method can also be easily applied to the analysis of any desired portion of a structure, such, for instance, as a continuous chord in a pin-connected bridge, or the loaded chord of a truss with subdivided panels.

The method of finding the amounts of the rotations of the various members from the Williot diagram can be used to advantage for the checking of changes in the angles in various parts of the structure, when the latter are computed by the method explained in "Modern Framed Structures," Grimm's book, and the A. R. E. A. report.

In order to use the method above proposed to the best advantage, the full panel loads for which the truss is designed should be employed, rather than arbitrary unit loads; for with the latter it will generally be impossible to judge from the approximate values of the T's as to the actual fibre stresses under the full panel loads, and it will be somewhat difficult to decide how accurately the work should be carried out. However, it can be used to determine the stresses due to loads on a portion of a truss only. When it is desirable to analyze a truss for partial loadings, as well as for full load over the entire span, the following plan should be followed:

First. Figure stresses for full dead plus live-plus-impact loads over the entire structure, and then compute therefrom (by proportion) the stresses for the dead load and the live-plus-impact load separately.

Second. Calculate the stresses for the live-plus-impact load at all except the end load point of the truss. It will be necessary to consider but three or four panels at the end of the span, as the secondary stresses in the other portions of the truss will be about the same as for the full load condition. The primary stresses and the elongations need be computed for the members in these panels only, and the Williot diagram can be started at the end of the truss. In figuring the values of the T's, it will be necessary to assume approximately their values at the points where the portion of the truss under consideration joins the remainder thereof; and these may be taken as equal to their values for full load, as previously figured, multiplied by the ratio of the primary chord stresses (at the said points) under partial loading to those under full loading.

Third. Repeat this method for as many loadings as may be desired, using each time one less panel load than before, and figuring each time the stresses in the panels near the "head of the train" only.

Quite a number of trusses have been analyzed for secondary stresses, and the results published. Grimm's book, "Modern Framed Structures," the A. R. E. A. report, and Mr. Kunz's article in *Engineering News* give a number of examples. In Figs. 11*a*, 11*c*, and 11*e* are shown the outlines of three riveted trusses which have been analyzed in the author's office. The first of these is a 200′ double track railway span, designed for Class R of *De Pontibus*; the second is a 201′ single track railway span of the Iowa Central Railway Bridge over the Mississippi River at Keithsburg, Ill.; and the third is a 428′ span employed in the first design of the Fratt Bridge over the Missouri River at Kansas City, Mo. The secondary stresses in the 200′ double track span are shown in Table 11*a* and Fig. 11*b;* those in the 201′ single track span, in Table 11*b* and Fig. 11*d;* and those in the 428′ span, in Table 11*c* and Fig. 11*f*. It is interesting to note the entirely different character of the stresses in the end panels of the trusses in Figs. 11*b* and 11*d*, which difference is due to the presence of the collision strut in the former. The secondary stresses in the trusses of the Fratt Bridge as finally constructed were not as high as those shown here, for some of the members were designed considerably shallower, and adjustments were made to relieve certain of the most severe secondary stresses. The large secondary stresses caused by the action of the members which are not a portion of the main truss system should be particularly observed.

In Table 11*g* and Fig. 11*j* are given the secondary stresses in the 296′ span previously discussed in this chapter.

In figuring the secondary fibre stresses in the trusses shown in Figs. 11*a*, 11*c*, 11*e*, and 11*g*, the gross moments of inertia were employed in all cases. While, strictly speaking, the net value should be used for the tension members, yet, as the higher stresses occur only at the panel points, where there is generally a considerable excess of section, the error in most cases is of small importance. Where there is no such excess, the net figures should be employed. In calculating the ratios of the secondary stresses to the primary stresses, the primary unit stresses were computed by using the gross areas of the members, so that the resulting percentages will be about correct for the tension members as well as for the compression members. It will be noted that the primary unit stresses in some of

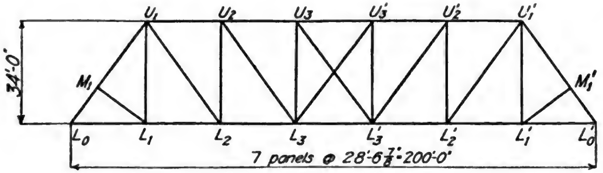


FIG. 11*a*. Truss Diagram of a 200-foot, Double-track-railway, Through, Riveted, Pratt-truss Span.

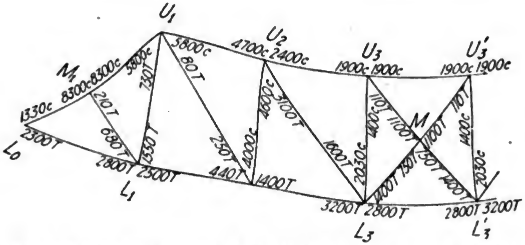


FIG. 11*b*. Secondary-stress Diagram of a 200-foot, Double-track-railway, Through, Riveted, Pratt-truss Span.

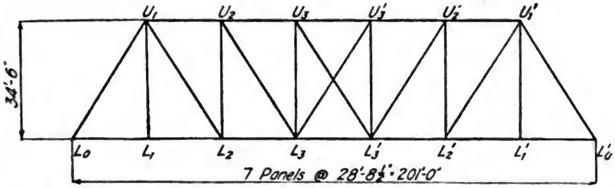


FIG. 11*c*. Truss Diagram of a 201-foot, Single-track-railway, Through, Riveted, Pratt-truss Span.

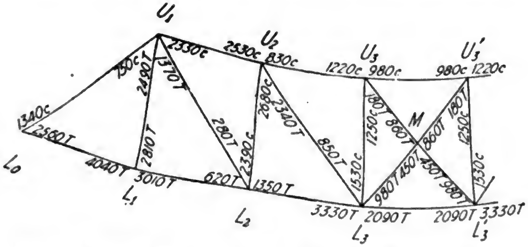


FIG. 11*d*. Secondary-stress Diagram of a 201-foot, Single-track-railway, Through, Riveted, Pratt-truss Span.

TABLE 11a—SECONDARY STRESSES IN A 200-FOOT, DOUBLE-

Member	Section	Gross Area	MAXIMUM PRI-
			Total
L_0M_1	1 Cov. Pl. 29 × ⅝ 2 $\angle^s$ 3½ × 3½ × ⅝ 2 $\angle^s$ 6 × 6 × $\frac{11}{16}$ 2 Bott. Pls. 7 × ⅝ 2 Webs 28¾ × $\frac{11}{16}$	90.1	1,328,000C
M_1U_1	Same as for L_0M_1	90.1	1,328,000C
U_1U_2	Same as for L_0M_1	90.1	1,425,000C
U_2U_3	Cov. Pls., $\angle^s$ etc. as for L_0M_1 2 Webs 28¾ × 1	108.0	1,707,000C
$U_3U'_3$	Same as for U_2U_3	108.0	1,707,000C
L_0L_1	4 $\angle^s$ 6 × 6 × ⅝ 2 Pls. 28 × ⅝	63.4	855,000T
L_1L_2	Same as for L_0L_1	63.4	855,000T
L_2L_3	4 $\angle^s$ 6 × 4 × ¾ 4 Pls. 28 × ¾	111.8	1,425,000T
$L_3L'_3$	Pls. and $\angle^s$ as for L_2L_3 2 Pls. 15½ × ½	127.3	1,707,000T
U_1L_2	4 $\angle^s$ 6 × 6 × ¾ 2 Pls. 24 × ¾	69.7	961,000T
U_2L_3	4 $\angle^s$ 3½ × 3½ × ½ 2 Pls. 21 × ⅞	49.8	617,000T
U_3M	2 $\sqsubset^s$ 15″ × 40 lbs.	23.5	319,000T
L_3M	2 $\sqsubset^s$ 15′ × 40 lbs.	23.5	319,000T
U_1L_1	2 $\sqsubset^s$ 15″ × 55 lbs.	32.4	410,000T
U_2L_2	4 $\angle^s$ 3½ × 3½ × ½ 2 Pls. 21 × ⅝	39.3	495,000C
U_3L_3	2 $\sqsubset^s$ 15″ × 40 lbs.	23.5	267,000C
M_1L_1	4 $\angle^s$ 6 × 4 × ⅜ 1 Pl. 18½ × ⅜	21.4	0

the web members are quite low, which is due to the fact that they were proportioned for reversal.

The stresses in the four trusses shown herewith, and those in most of the others which have been published, have been figured for a single position of loading, generally full load over the entire span. While these are not the maximum secondary stresses in all portions of the structure, they are usually pretty close to the greatest values for most of the members. As a general rule, the maximum total stress on any member occurs when it receives its greatest direct stress, even though its secondary stress may be greater for some other condition of loading. Evidently, there-

Track-Railway, Through, Riveted, Pratt-Truss Span

mary Stresses	Secondary Stresses			
Unit	Point	Top	Bottom	Per Cent of Maximum Primary
14,850C	L_0	1,330C	......	9.0
	M_1	8,300C	......	55.9
14,850C	M_1	8,300C	......	55.9
	U_1	......	5,800C	39.0
15,820C	U_1	......	5,800C	36.6
	U_2	4,700C	......	30.3
15,800C	U_2	2,400C	......	15.2
	U_3	1,900C	......	12.0
15,800C	U_3	1,900C	......	12.0
	U'_3	1,900C	......	12.0
13,480T	L_0	......	2,300T	17.1
	L_1	......	2,800T	20.8
13,480T	L_1	......	2,500T	18.5
	L_2	440T	......	3.3
12,740T	L_2	1,400T	......	11.0
	L_3	......	3,200T	25.1
13,420T	L_3	......	2,800T	20.9
	L'_3	......	2,800T	20.9
13,780T	U_1	80T	......	0.6
	L_2	......	250T	1.8
12,390T	U_2	......	3,100T	25.0
	L_3	1,600T	......	12.9
13,570T	U_3	......	110T	0.8
	M	......	1,100T	8.1
13,570T	L_3	......	1,400T	10.3
	M	......	750T	5.5
		Left	Right	
12,650T	L_1	......	1,550T	12.3
	U_1	730T	......	5.8
12,600C	L_2	4,000C	......	31.7
	U_2	......	4,600C	36.5
11,260C	L_3	2,030C	......	18.0
	U_3	......	1,400C	12.4
		Top	Bottom	
0	M_1	210T	......	......
	L_1	......	680T	......

fore, the results for a fully loaded span represent the worst effects on the chords and end posts. For the web members near the ends of the truss, this same loading also gives fair values. For web members near the centre of the truss, the correct maxima may be considerably greater than those given by the said full loading; but as these members are generally quite slender, and frequently have an excess of section, the secondary stresses therein will rarely be of any importance.

Quite a number of valuable general conclusions can be drawn from the investigations thus far made. One of the best summaries is that

TABLE 11*b*—SECONDARY STRESSES IN A 201-FOOT, SINGLE-

Member	Section	Gross Area	MAXIMUM PRI-
			Total
L_0U_1.......	1 Cov. Pl. 27 × $\frac{7}{16}$ 4 ∠^{s} 3½ × 3½ × $\frac{3}{8}$ 2 ∠^{s} 6 × 4 × $\frac{5}{8}$ 2 Webs 20 × $\frac{11}{16}$	60.95	769,000*C*
U_1U_2.......	Cov. Pl. and ∠^{s} as for L_0U_1 2 Webs 20 × $\frac{9}{16}$	55.95	820,000*C*
U_2U_3.......	Cov. Pl. and ∠^{s} as for L_0U_1 2 Webs 20 × $\frac{13}{16}$″	65.95	984,000*C*
U_3U_3'......	Same as for U_2U_3	65.95	984,000*C*
L_0L_1........	4 ∠^{s} 3½ × 3½ × $\frac{7}{16}$ 2 Pls. 24 × $\frac{9}{16}$	38.48	492,000*T*
L_1L_2........	4 ∠^{s} 3½ × 3½ × $\frac{7}{16}$ 2 Pls. 24 × $\frac{9}{16}$	38.48	492,000*T*
L_2L_3........	4 ∠^{s} 3½ × 3½ × ½ 4 Pls. 24 × ½	61.00	820,000*T*
$L_3L'_3$.......	4 ∠^{s} 3½ × 3½ × ½ 4 Pls. 24 × $\frac{5}{8}$	73.00	984,000*T*
U_1L_2.......	4 ∠3^s ½ × 3½ × ½ 2 Pls. 20 × $\frac{3}{4}$	43.00	554,000*T*
U_2L_3.......	4 ∠^{s} 3½ × 3½ × $\frac{7}{16}$ 2 Pls. 20 × ½	31.48	357,000*T*
U_3M.......	4 ∠^{s} 6 × 3½ × $\frac{7}{16}$	15.88	184,000*T*
L_3M.......	4 ∠^{s} 6 × 3½ × $\frac{7}{16}$	15.88	184,000*T*
U_1L_1.......	2 [^{s} 15″ × 33 lbs.	19.80	226,000*T*
U_2L_2.......	2 [^{s} 15″ × 45 lbs.	26.50	288,000*C*
U_3L_3.......	2 [^{s} 15″ × 33 lbs.	19.80	155,000*C*

given in the report of the A. R. E. A. Committee before mentioned, from which the following extract is taken:

"(1) In any given truss the amount of bending, or the sharpness of curvature, produced in the members is, in general, proportional to the intensity of the primary stresses, that is, the larger the primary stresses the greater the deformation, both longitudinally and in bending.

"The fibre stresses resulting from this bending, that is, the secondary unit stresses, are proportional to the bending and, therefore, proportional to the primary stresses. It follows that in any given truss, for any given method of loading, the secondary stresses bear a fixed percentage to the primary stresses, no matter what the amount of the load may be.

"(2) Other things being equal, or similar, the percentage of the secondary stress is proportional to the distance to outer fibre in the plane of bending, and inversely proportional to the lengths of the members. When the members are symmetrical the

TRACK-RAILWAY, THROUGH, RIVETED, PRATT-TRUSS SPAN

mary Stresses	Secondary Stresses			
Unit	Point	Top	Bottom	Per Cent of Maximum Primary
12,620C	L_0	1,340C	......	10.6
	U_1	......	750C	5.9
14,640C	U_1	......	2,330C	15.9
	U_2	2,530C	......	17.3
14,920C	U_2	830C	......	5.6
	U_3	1,220C	......	8.2
14,920C	U_3	980C	......	6.6
	U'_3	980C	......	6.6
12,640T	L_0	2,560T	......	20.2
	L_1	......	4,040T	32.0
12,640T	L_1	......	3,010T	23.8
	L_2	620T	......	4.9
13,440T	L_2	1,350T	......	10.0
	L_3	......	3,330T	24.8
13,490T	L_3	......	2,090T	15.5
	L'_3	......	2,090T	15.5
12,650T	U_1	......	1,370T	10.8
	L_2	280T	......	2.2
11,330T	U_2	......	2,340T	20.7
	L_3	850T	......	7.5
11,570T	U_3	......	180T	1.6
	M	......	860T	7.4
11,570T	L_3	......	980T	8.5
	M	......	450T	3.9
		Left	Right	
11,410T	L_1	......	2,810T	24.6
	U_1	2,490T	......	21.8
10,860C	L_2	2,390C	......	22.0
	U_2	......	2,680C	24.7
7,870C	L_3	1,530C	......	19.5
	U_3	......	1,250C	15.9

secondary stresses will be proportional to the ratios of widths to lengths. Thus, if two trusses are compared in which the general dimensions and moments of inertia of members are proportional, but the ratio of depth to length of the various members of one truss is in all cases twice this ratio in the other truss, then the percentages of the secondary stresses in the first truss will be twice the percentages in the second truss. This relation comes about from the fact that in the two assumed cases, if the primary stresses are equal, the *angular* deformations of the members in the two trusses will be the same, and, for a given angular deformation, the resulting fibre stress is proportional to the ratio of depth to length.

"(3) The secondary stresses in any particular member are dependent upon the distortions of all the members of the truss, but, primarily, upon the distortions of the members of the particular triangles of which this member is a part and of the members of the adjoining triangles.

"(4) Bearing in mind the above principles, it is possible to predict from calculations of typical trusses the secondary stresses in any particular type of truss in terms of ratio of depths to lengths of members with a considerable degree of accuracy.

"(5) The more uniform the proportions of a truss the less, in general, will be the secondary stresses. Sudden changes in length, width, or moment of inertia are likely to result in relatively large secondary stresses.

"(6) Trusses consisting of approximately equilateral triangles, and without hangers or vertical struts, present the most uniform conditions and will have, in general, the lowest secondary stresses. A truss composed of right-angle triangles will show somewhat higher secondary stresses, and such stresses will be large if the ratio of height to panel length is large.

"(7) Wherever hangers or vertical struts are used to support single joint loads, as in a Warren girder with verticals, or in a Pratt truss (at the hip vertical, or at the centre vertical in the case of a deck bridge) the secondary stresses in the adjacent chord members are likely to be considerably larger than elsewhere. The best arrangement, so far as secondary stresses are concerned, is where each web member forms an integral part of the entire truss so that its stress will gradually change as the load progresses.

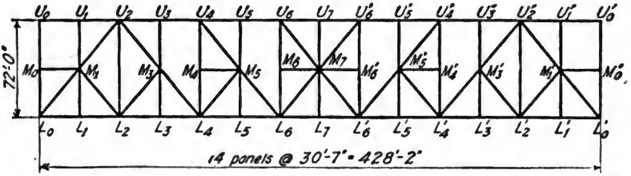


FIG. 11e. Truss Diagram of a 428-foot, Double-deck, Double-track-railway, Electric-railway, and Highway, Riveted, Petit-truss Span.

"(8) Considering the fact that secondary stresses are, in general, proportional to the ratios of depths to lengths and considering the principles stated in the preceding paragraphs, it follows that the secondary stresses in trusses where the panels are subdivided, as in the Baltimore or Petit system, are likely to be very high. In the case of pin-connected trusses this may also be the case with the top chord."

Several other conclusions can be stated, more or less in line with those quoted.

The double-intersection triangular truss without verticals is found to give exceedingly high secondary stresses. If verticals be added at each panel point, these stresses reduce very materially and become of about the same magnitude as those in the Pratt and the Triangular systems.

The K-system of trussing, which is being used for the new Quebec Bridge, is quite free from severe secondary stresses, being similar to the Pratt and the Triangular systems in this respect, but permitting economical panel-lengths with great truss depths as in the Petit system. The fact of its having no secondary members explains its immunity from excessive secondary stresses.

The conclusion can also be drawn from the A. R. E. A. report that for trusses of the Triangular or the Pratt type without hangers, the ratio of secondary stresses to the primary ones will probably run about 3.5 × $\frac{\text{depth of member}}{\text{length of member}}$; but if hangers be used, such as the hip vertical in a

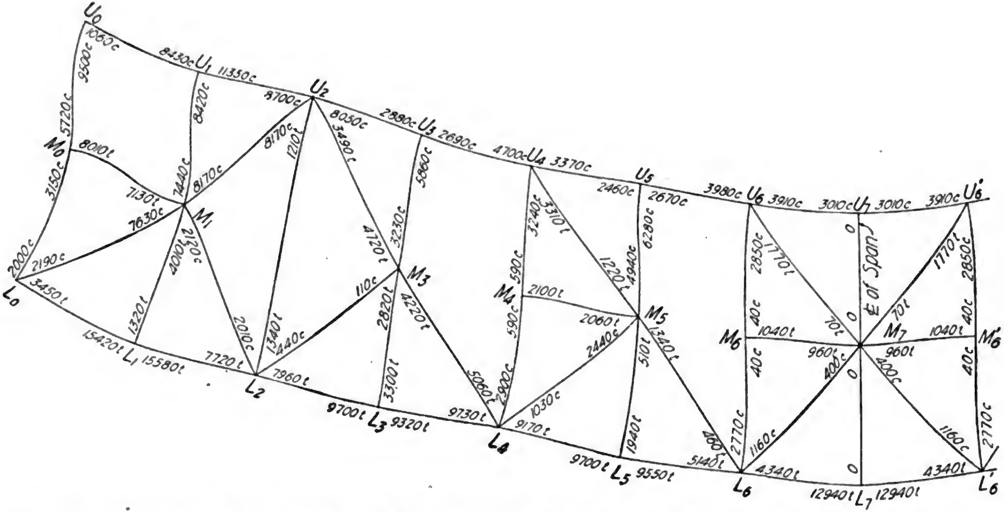


FIG. 11*f*. Secondary-stress Diagram of a 428-foot, Double-deck, Double-track-railway, Electric Railway, and Highway, Riveted, Petit-truss Span.

TABLE 11c.—SECONDARY STRESSES IN A 428-FOOT, DOUBLE-DECK, PETIT-TRUSS

Member	Section	Gross Area	MAXIMUM PRI-
			Total
U_0U_1	8 $\angle^s$ 6 × 6 × $\frac{7}{16}$ 2 Webs 54 × $\frac{3}{8}$	81.0	280,000*C*
U_1U_2	Same as for U_0U_1	81.0	237,000*C*
U_2U_3	2 Cov. Pls. 61 × $\frac{13}{16}$ 4 $\angle^s$ 6 × 6 × $\frac{3}{4}$ 4 $\angle^s$ 8 × 6 × $\frac{1}{2}$ 2 Webs 54 × $\frac{3}{4}$ 2 Webs 54 × $\frac{11}{16}$	315.1	5,234,000*C*
U_3U_4	Same as for U_2U_3	315.1	5,191,000*C*
U_4U_5	Cov. Pls. and $\angle^s$ as for U_2U_3 4 Webs 54 × $\frac{5}{8}$ 2 Webs 54 × $\frac{11}{16}$	369.1	6,157,000*C*
U_5U_6	Same as for U_4U_5	369.1	6,114,000*C*
U_6U_7	Cov. Pls. and $\angle^s$ as for U_2U_3 4 Webs 54 × $\frac{5}{8}$ 2 Webs 54 × $\frac{11}{16}$	375.8	6,196,000*C*
L_0L_1	4 $\angle^s$ 6 × 6 × $\frac{7}{8}$ 4 Pls. 72 × $\frac{11}{16}$	237.0	3,275,000*T*
L_1L_2	Same as for L_0L_1	237.0	3,275,000*T*
L_2L_3	Same as for L_0L_1	237.0	3,275,000*T*
L_3L_4	Same as for L_0L_1	237.0	3,275,000*T*
L_4L_5	4 $\angle^s$ 6 × 6 × $\frac{7}{8}$ 4 Pls. 72 × $\frac{3}{4}$ 2 Pls. 72 × $\frac{11}{16}$	372.0	5,295,000*T*
L_5L_6	Same as for L_4L_5	372.0	5,295,000*T*
L_6L_7	4 $\angle^s$ 6 × 6 × $\frac{7}{8}$ 8 Pls. 72 × $\frac{11}{16}$	435.0	6,174,000*T*
L_0M_1	Cov. Pls. and $\angle^s$ as for U_2U_3 4 Webs 54 × $\frac{3}{4}$	321.9	5,054,000*C*
M_1U_2	Cov. Pls. and $\angle^s$ as for U_2U_3 2 Webs 54 × $\frac{3}{4}$ 2 Webs 54 × $\frac{1}{2}$	294.9	4,665,000*C*
U_2M_3	4 $\angle^s$ 6 × 6 × $\frac{5}{8}$ 6 Pls. 48 × $\frac{11}{16}$	226.4	3,234,000*T*
M_3L_4	4 $\angle^s$ 6 × 6 × $\frac{5}{8}$ 2 Pls. 48 × $\frac{1}{2}$ 4 Pls. 48 × $\frac{11}{16}$	208.4	2,939,000*T*
U_4M_5	4 $\angle^s$ 6 × 6 × $\frac{5}{8}$ 2 Pls. 36 × $\frac{3}{4}$ 2 Pls. 36 × $\frac{11}{16}$	131.9	1,868,000*T*
M_5L_6	4 $\angle^s$ 6 × 6 × $\frac{5}{8}$ 2 Pls. 36 × $\frac{11}{16}$ 2 Pls. 36 × $\frac{1}{2}$	113.9	1,618,000*T*

Double-Track Railway, Electric-Railway, and Highway, Riveted Span

mary Stresses	Secondary Stresses			
Unit	Point	Top	Bottom	Per Cent of Maximum Primary
3,460C	U_0	....	1,060C	....
	U_1	8,430C	....	....
2,930C	U_1	11,350C	....	....
	U_2	....	8,700C	....
16,610C	U_2	....	8,050C	48.4
	U_3	2,880C	....	17.3
16,480C	U_3	2,690C	....	16.3
	U_4	4,700C	....	28.5
16,660C	U_4	3,370C	....	20.2
	U_5	....	2,460C	14.7
16,580C	U_5	....	2,670C	16.1
	U_6	3,980C	....	24.0
16,480C	U_6	3,910C	....	23.7
	U_7	3,010C	....	18.3
13,830T	L_0	3,450T	....	25.0
	L_1	....	15,420T	111.6
13,830T	L_1	....	15,580T	112.7
	L_2	7,720T	....	55.8
13,830T	L_2	7,960T	....	57.6
	L_3	....	9,700T	70.1
13,830T	L_3	....	9,320T	67.4
	L_4	9,730T	....	70.4
14,230T	L_4	9,170T	....	64.5
	L_5	....	9,700T	68.2
14,230T	L_5	....	9,550T	67.1
	L_6	5,140T	....	36.1
14,200T	L_6	4,340T	....	30.6
	L_7	....	12,940T	91.1
16,200C	L_0	2,190C	....	13.5
	M_1	7,630C	....	47.1
15,840C	M_1	8,170C	....	51.6
	U_2	....	8,170C	51.6
14,280T	U_2	3,490T	....	24.5
	M_3	....	4,720T	33.1
14,100T	M_3	....	4,220T	30.0
	L_4	5,060T	....	35.9
14,170T	U_4	....	3,310T	23.4
	M_5	1,220T	....	8.6
14,210T	M_5	1,340T	....	9.4
	L_6	....	460T	3.3

TABLE 11c.—*Continued.*—SECONDARY STRESSES IN A 428-FOOT, DOUBLE-PETIT-TRUSS

Member	Section	Gross Area	MAXIMUM PRI-
			Total
U_6M_7.......	4 $\angle^s$ 6 × 4 × ½ 2 Pls. 22 × $\frac{9}{16}$	43.7	497,000T
L_6M_7.......	Same as for U_6M_7	43.7	497,000C
M_1L_2.......	Same as for U_6M_7	43.7	522,000C
L_2M_3.......	Same as for U_6M_7	43.7	522,000C
L_4M_5.......	Same as for U_6M_7	43.7	522,000C
U_2L_2.......	4 $\angle^s$ 6 × 6 × ⅝ 2 Pls. 36 × $\frac{9}{16}$ 2 Pls. 36 × ½	104.9	1,325,000T
L_4M_4.......	4 $\angle^s$ 6 × 6 × ⅝ 4 Pls. 36 × $\frac{9}{16}$	109.4	1,540,000C
M_4U_4.......	Same as for L_4M_4	109.4	1,540,000C
L_6M_6.......	4 $\angle^s$ 6 × 6 × $\frac{7}{16}$ 2 Pls. 26 × $\frac{9}{16}$	49.5	492,000C
M_6U_6.......	Same as for L_6M_6	49.5	492,000C
L_0M_0.......	4 $\angle^s$ 3½ × 3½ × ⅜ 2 Pls. 20 × ½	29.9	370,000C
M_0U_0.......	Same as for L_0M_0	29.9	370,000C
L_1M_1.......	4 $\angle^s$ 6 × 4 × ½ 2 Pls. 26 × ¾	58.0	682,000T
M_1U_1.......	4 $\angle^s$ 3½ × 3½ × ⅜ 2 Pls. 24 × ½	33.9	125,000C
L_3M_3.......	Same as for L_1M_1	58.0	682,000T
M_3U_3.......	Same as for M_1U_1	33.9	115,000C
L_5M_5.......	Same as for L_1M_1	58.0	682,000T
M_5U_5.......	Same as for M_1U_1	33.9	115,000C
L_7M_7.......	Same as for L_1M_1	58.0	682,000T
M_7U_7.......	Same as for M_1U_1	33.9	115,000C
M_0M_1......	2 ⊏^{s} 12″ × 25 lbs.	14.7	0
M_4M_5......	2 ⊏^{s} 12″ × 25 lbs.	14.7	0
M_6M_7......	2 ⊏^{s} 12″ × 25 lbs.	14.7	0

DECK, DOUBLE-TRACK RAILWAY, ELECTRIC-RAILWAY, AND HIGHWAY, RIVETED SPAN

MARY STRESSES	SECONDARY STRESSES			
Unit	Point	Top	Bottom	Per Cent of Maximum Primary
11,370*T*	U_6	....	1,770*T*	15.6
	M_7	....	70*T*	0.6
11,370*C*	L_6	1,160*C*	....	10.2
	M_7	400*C*	....	3.5
11,940*C*	M_1	....	2,120*C*	17.8
	L_2	2,010*C*	....	16.8
11,940*C*	L_2	440*C*	....	3.7
	M_3	110*C*	....	0.9
11,940*C*	L_4	....	1,030*C*	8.6
	M_5	2,440*C*	....	20.4
		Left	Right	
12,630*T*	L_2	....	1,340*T*	10.6
	U_2	1,210*T*	....	9.6
14,060*C*	L_4	2,900*C*	....	20.6
	M_4	590*C*	....	4.2
14,060*C*	M_4	590*C*	....	4.2
	U_4	....	3,240*C*	23.1
9,950*C*	L_6	2,770*C*	....	27.9
	M_6	....	40*C*	0.4
9,950*C*	M_6	....	40*C*	0.4
	U_6	....	2,850*C*	28.7
12,360*C*	L_0	2,000*C*	....	16.2
	M_0	3,150*C*	....	25.5
12,360*C*	M_0	5,720*C*	....	46.3
	U_0	....	9,500*C*	76.8
11,760*T*	L_1	1,320*T*	....	11.2
	M_1	....	4,010*T*	34.1
3,860*C*	M_1	7,440*C*	....	....
	U_1	....	8,420*C*	....
11,760*T*	L_3	....	3,300*T*	28.1
	M_3	2,820*T*	....	24.0
3,390*C*	M_3	3,230*C*	....	....
	U_3	....	5,860*C*	....
11,760*T*	L_5	....	1,940*T*	16.5
	M_5	....	510*T*	4.3
3,390*C*	M_5	4,940*C*	....	....
	U_5	....	6,280*C*	....
11,760*T*	L_7	0	....	0
	M_7	0	....	0
3,390*C*	M_7	0	....	0
	U_7	0	....	0
		Top	Bottom	
0	M_0	8,010*T*	7,130*C*	....
	M_1	8,010*C*	7,130*T*	....
0	M_4	2,100*T*	....	....
	M_5	....	2,060*T*	....
0	M_6	1,040*T*	....	....
	M_7	....	960*T*	....

Pratt truss, or verticals at every panel-point in the Warren system, the ratio for the loaded chord is likely to be $5 \times \frac{\text{depth of member}}{\text{length of member}}$. The effect of hangers in a Petit truss is especially great, because their adoption is very likely to be accompanied by the employment of unusually high values of $\frac{\text{depth of member}}{\text{length of member}}$. It should be noted in particular that when sub-ties are used in the through Petit truss, the secondary stresses in the bottom chord are two or three times as great as those occurring there when sub-struts are employed.

The effect of pins on the secondary stresses deserves particular attention. For eye-bars and narrow riveted members, such as four-angle I-struts, the secondary stresses will not usually develop sufficient moment to cause the members to turn on the pins. However, on account of the low ratios of $\frac{\text{depth of member}}{\text{length of member}}$ in such pieces, the secondary stresses therein are not likely to be high; and it is further probable that they become zero under dead load. Where deep riveted members are connected to pins, rotation may be expected to take place when the secondary stresses become ten (10) or fifteen (15) per cent of the primary ones, if the entire stress in the member is carried by the pin; and these percentages will be correspondingly reduced if only a portion of the stress is carried by the pin, as in the case of a continuous top chord. The maximum secondary stresses in such members will, therefore, rarely exceed ten (10) or fifteen (15) per cent of the primary ones. In analyzing a truss having pin-connections by the method previously presented in this chapter, the first trial values of the T's should be computed on the assumption that all members are fixed to the pins. The moments tending to turn the pins in the deep riveted members can then be calculated, using these first trial values of the T's; and from the said values it can be determined with sufficient accuracy whether the said members will turn. If any member be found to turn, the moment on the end thereof should be taken equal to the moment which will just cause rotation. In the case of a continuous chord which turns on a pin, this critical value of the moment will be applied by the chord to the other members, and by the other members to the continuous chord. The method of figuring the turning moment which is required to overcome pin-friction is discussed on page 464 of "Modern Framed Structures," Part II, as is also the question of the effect of pin-connections in general.

Perhaps the most important feature of the problem of secondary stresses is the determination of what effects can be avoided and what can not. It has been noted already that in the Pratt, Triangular, and K-trusses, the secondary stresses due to the action of the main truss members are small, while those due to secondary members are likely to be quite large. Evidently the important point is to get rid of the effects of the secondary members.

The possibility of the reduction in the stresses due to the action of the main members will first be considered. It is clear that these members, other than chords continuous over two or more panels, will be practically free from secondary stresses at the time they are riveted up, which means under the no-load or partial-dead-load condition. A chord can be relieved by cambering the truss for dead load plus all or a part of the live and impact loads, laying the said chord out straight, riveting it up, and then bending it to the proper camber curve before attaching the web members. The only way in which anything can be accomplished in relieving the web members is by putting the span under a good portion of its total load before the riveting is done. This could be effected by connecting the various web members to the gussets by drift-pins and bolts, and doing the riveting after the span has been swung and loaded. The drift-pins should be driven in one compact group at the centre of the connection. Another method would be to provide pins to carry the dead-load stresses at the points where the worst secondary stresses occur, and rivet up these points only after the span has been swung. However, as the secondary stresses involved are not usually large, it would rarely be worth while to go to that expense.

The stresses due to the action of the secondary members are quite large, however; and they should be reduced, if practicable. Evidently the objectionable feature of these members is that they pull the main members decidedly out of line; and this effect is to be avoided as far as possible. For a member which carries no stress, the problem is simple. The proper length when the truss is loaded by the dead plus a part or all of the live-and-impact loads is determined by means of a Williot diagram and the holes in one end are either left blank or are sub-punched and not reamed. The main connections and that in one end of the sub-member are fully riveted and the span is swung, and then a heavy load is placed on the bridge. While it is thus loaded, the holes in the other end of the sub-member are to be reamed or drilled and then the rivets are to be driven.

When secondary members carry direct stresses, however, as do hip verticals, hangers, or sub-struts, this procedure cannot be followed, because their connections must be riveted before any load can be placed on them, and preferably before the span is swung. The following method is, therefore, suggested. The deformations of all of the members of the truss under dead load plus a part or all of the live-and-impact loads are to be calculated, the tension members are to be shortened and the compression members lengthened by these amounts, and the proper positions of all panel-points under no-load are to be computed. Then the truss is to be laid out in the shops with all of the main panel-points occupying these positions, and with all main members straight between the main panel-points. All the connections in all the main members are then to be reamed, and also the connection at one end of each sub-member. In order to ream the other end connection of each sub-member so that the piece will be of proper length, the main members would have to be bent and the intermediate panel-points shifted. As this would be very difficult to do, and as the results would probably be inaccurate, the amount that each panel-point should be shifted with respect to any adjacent point is to be figured, and then the already reamed connection of the sub-member joining these two panel-points is to be unbolted, and the member slipped longitudinally by the amount that the panel-point should be moved. The connection at the other end of the member is then to be reamed. This procedure can, of course, be followed with sub-members which have no stress as well as with those having stress in them. When the truss is erected, the main panel-points are to be put in their proper position on the camber-blocking with the members straight between the said panel-points; and then the connections for the main members and for one end of each sub-member are to be riveted up. The main members are then to be bent until the holes in the other ends of the sub-members match, when they also are to be riveted up. This bending may be accomplished by the use of jacks or steamboat-rachets, by starting tapered drift pins in several holes and gradually driving them home, and by lowering the main panel-points a trifle on their blocking. When the truss is swung and covered with the loading for which the deformations of the members were figured, it will be free from all secondary stresses due to the action of the sub-members.

It would appear at first thought that it would be best to correct for secondary stresses in such a manner that they will be zero when the full dead-plus-live-plus-impact loads are on the span. However, at the instant that the chords have their maximum stresses, the live-load stress in any sub-member may be much less than its maximum value, or even nearly zero. Since it is for the effect of these members that we wish to correct, it will probably give just as satisfactory results if the adjustments are made in such a manner that the effect of the said secondary members will be eliminated when the span is under dead load plus one-half of live-and-impact loads.

When a riveted simple truss span is erected by semi-cantilevering, particular attention should be given to the secondary stresses, because the change in the primary stresses from erection conditions to full load conditions is very great in some members. It may be desirable to introduce temporary pins into some of the connections, or to use drift-pins only for some of them until after the truss is acting as a simple span.

The angle of rotation of members at a support is usually quite large, so that the use of pins at these points is almost imperative for spans of any size. This applies to cantilever bridges as well as to simple truss spans, pin-bearings being advisable at the main pier; for, otherwise, the stresses due to movement of the members must be considered and provided for.

Clause 90 of Chapter LXXIX, entitled "Correction of Secondary Stresses," reads as follows:

> "The secondary stresses in riveted trusses are to be modified by lengthening and shortening the various truss members the amounts of their respective shortening and lengthening under dead load plus one half of the live-plus-impact load, drilling or reaming the chord splices while the chords are assembled in straight lines, then forcing the truss members into their proper positions for connection to each other before drilling or reaming the holes in the joints."

This clause was submitted for comment in person by the author to Paul L. Wolfel, Esq., C.E., chief engineer of the McClintic-Marshall Construction Company, who is, undoubtedly, the highest American authority upon the subject of the correction of secondary stresses. After reading it several times and considering it carefully, he reported that while it might be difficult or even impracticable fully to live up to it in some extreme cases, it covered the ground so thoroughly that he could not suggest any improvement in its wording. It will be noted that there is no essential difference between this clause and the method of correction of secondary stresses previously presented in this chapter.

BENDING STRESSES IN THE PLANE OF THE TRUSS DUE TO ECCENTRICITY, TO WEIGHT OF MEMBERS, AND TO TRANSVERSE LOADS ON MEMBERS

These bending stresses are not secondary stresses in just the sense that the word is generally used; but as they require for their complete analysis the principles employed in secondary stress computations, they will be considered briefly here. It will first be necessary to state a few formulæ for continuous beams.

Suppose a beam joining any two points 1 and 2 be acted upon by a moment M_1 at the point 1, causing the end tangent at 1 to turn through an angle T_1. Then the value of T_1 will depend upon the condition of the far end 2, the length of the member l, and its moment of inertia I. We shall consider four conditions:

First. Beam fixed in direction at 2, so that T_2 (the angle through which the end tangent at 2 turns) equals zero.

We have from Equation 1 of this chapter.

$$M_1 = \frac{4\,EIT_1}{l}, \qquad \text{[Eq. 7]}$$

and

$$T_1 = \frac{M_1\,l}{4\,EI}, \qquad \text{[Eq. 8]}$$

whence

$$f_1 = \frac{4\,EcT_1}{l}. \qquad \text{[Eq. 9]}$$

We also have $M_2 = \dfrac{2\,EIT_1}{l} = \dfrac{M_1}{2}$, [Eq. 10]

and $f_2 = \dfrac{2\,EcT_1}{l}$. [Eq. 11]

In the above equations

f_1 = stress on extreme fibre of the member at 1,
f_2 = stress on extreme fibre of the member at 2,
and M_2 = moment at 2.

Second. Beam continuous at 2, and over several supports beyond.

We have from the Theorem of Three Moments,

$$M_2 = 0.27M_1. \qquad \text{[Eq. 12]}$$

From this we find, from Equation 1:

$$T_2 = -\ 0.27T_1. \qquad \text{[Eq. 13]}$$

$$\therefore M_1 = \frac{3.46\,EIT_1}{l}, \qquad \text{[Eq. 14]}$$

$$T_1 = \frac{0.29\,M_1\,l}{E\,I}, \qquad \text{[Eq. 15]}$$

$$f_1 = \frac{3.46\,EcT_1}{l}, \qquad \text{[Eq. 16]}$$

$$M_2 = \frac{0.94\,EIT_1}{l}, \qquad \text{[Eq. 17]}$$

$$T_2 = -\ \frac{0.08\,M_1\,l}{E\,I}, \qquad \text{[Eq. 18]}$$

and

$$f_2 = \frac{0.94\,EcT_1}{l}. \qquad \text{[Eq. 19]}$$

Third. Beam simply supported at 2.

Since in this case M_2 is zero, we have, by Equation 1,

$$T_2 = -\ \frac{1}{2}\,T_1. \qquad \text{[Eq. 20]}$$

$$\therefore M_1 = \frac{3\,EIT_1}{l}, \qquad \text{[Eq. 21]}$$

$$T_1 = \frac{M_1\,l}{3\,E\,I}, \qquad \text{[Eq. 22]}$$

$$f_1 = \frac{3\,EcT_1}{l}, \qquad \text{[Eq. 23]}$$

and

$$T_2 = \frac{M_1\,l}{6\,E\,I}. \qquad \text{[Eq. 24]}$$

Fourth. Beam partly restrained at 1 by other beams, and fixed at 2, carrying a uniform load of p per lineal unit.

We have from Equation 28 on p. 469 of "Modern Framed Structures," Part II,

$$M_1 = \frac{4\,EIT_1}{l} \pm \frac{p\,l^2}{12}, \qquad \text{[Eq. 25]}$$

$$T_1 = \frac{M_1\,l}{4\,EI} \mp \frac{p\,l^3}{48\,EI}, \qquad \text{[Eq. 26]}$$

$$f_1 = \frac{4\,EcT_1}{l} \pm \frac{p\,l^2 c}{12\,I}, \qquad \text{[Eq. 27]}$$

$$M_2 = \frac{2\,EIT_1}{l} \mp \frac{p\,l^2}{12}, \qquad \text{[Eq. 28]}$$

$$f_2 = \frac{2\,EcT_1}{l} \mp \frac{p\,l^2 c}{12\,I} \qquad \text{[Eq. 29]}$$

The upper of the two signs before the second term of the right-hand member of each equation is to be used when Point 1 is at the left end of the beam, and Point 2 at the right; while the lower sign is to be used when the reverse is true.

The effect of eccentricity at any joint can be estimated as follows: Figure the moment M at the point due to the eccentricity. This moment will cause the ends of all the members meeting at the point to turn through some angle T_1. We may then write

$$M = \Sigma\,\frac{KEIT_1}{l}, \qquad \text{[Eq. 30]}$$

where K depends upon the condition of the other end of each member. Ordinarily, this will vary from the continuous condition to one nearly fixed. The most probable values of K should be selected, using Equations 7, 14, and 21. The value of T_1 can thus be calculated, and then the moment M_1 and fibre stress f_1 in each member. It will rarely be worth while to consider the effect of eccentricity except at the point where it occurs. If such be deemed necessary, however, the method given on page 462 of "Modern Framed Structures," Part II, can be followed.

The bending due to the weight of a member is generally to be determined by the approximate formula given in the specifications of Chapter LXXVIII. If an exact figure be desired, the method outlined in Arts. 330 and 331 of "Modern Framed Structures," Part II, can be employed.

The bending due to transverse loads on a member is also usually to be computed by the same approximate formula just mentioned. When all the panels of a chord which is subjected to such loads are fully loaded, there will be little tendency to rotate at any of the panel-points, or to set up secondary stresses in the truss; but if only a portion of the chord is loaded, the gusset-plates at the panel-point near the head of the train may be rotated somewhat, thus causing secondary stresses in all of the members meeting at the said point. This condition may be analyzed as follows: The chord loaded is to be assumed as partly restrained at the panel-point in question and fixed at its other end, as was assumed for the fourth condition given above. The condition of fixity at the other point will be about realized, as there will be a load on the next panel also, causing the end tangent there to remain nearly in its original position. The moment M_1 in the chord at the point in question is equal to the sum of the moments in the other truss members meeting at this point, hence we may write,

$$\Sigma \frac{K\,EIT_1}{l} = \frac{4\,EIT_1}{l} \mp \frac{Pl^2}{12}. \qquad \text{[Eq. 31]}$$

The summation in this equation is taken for all the members meeting at the point except the chord member which is loaded; and K has the same significance as in Equation 30. The value of K for each member is estimated, and T_1 is then calculated. From this the values of M_1 and f_1 for each member can be easily figured. The effects on the far ends of the various members will rarely need consideration.

The secondary stresses due to eccentricity and to transverse loads, as calculated approximately above, are in addition to the secondary stresses due to rigidity of the joints. Their effects will be small in almost any well designed truss.

BENDING STRESSES IN MEMBERS OF A TRANSVERSE FRAME DUE TO THE DEFLECTION OF FLOOR-BEAMS AND TO PRIMARY STRESSES IN POSTS

These stresses result from the fact that the floor-beams and the posts are riveted rigidly to each other. It is found by an analysis of the problem that the floor-beams are only slightly fixed at the ends, but that the bending stresses in the posts are considerable. The analysis is given in "Modern Framed Structures," Part II, page 502. If in deriving Equations 45 and 46 on that page we take $a = 0.3b$, which is more nearly correct than $a = \frac{1}{4}b$, we get

$$\frac{f_2}{f_1} = \frac{b}{h} \times \frac{c_2}{c_1} \qquad \text{[Eq. 32]}$$

for the condition of verticals hinged at the top, and

$$\frac{f_2}{f_1} = \frac{4}{3} \times \frac{b}{h} \times \frac{c_2}{c_1} \qquad \text{[Eq. 33]}$$

for verticals fixed at top. In these equations

f_1 = fibre stress in floor-beam at centre,

f_2 = fibre stress in post at bottom,
c_1 = depth of floor-beam,
c_2 = width of post,
b = distance from centre to centre of trusses,
and h = distance from centre line of floor-beam to centre line of overhead bracing.

Probably the best results will be obtained by using Equation 32, and taking h as the distance from centre line of floor-beam to centre line of bottom strut of sway bracing. For single-track railway bridges of short span, fair values of these quantities will be

f_1 = 14,000 lbs. per sq. in.
$b = 17'$
$h = 21'$
$c_1 = 54''$
$c_2 = 13''$

$$\therefore f_2 = 14{,}000 \times \frac{13}{54} \times \frac{17}{21} = 2{,}730 \text{ nearly.}$$

For long-span, single-track railway bridges we shall have,

f_1 = 14,000 lbs. per sq. in.
$b = 17.5'$
$h = 26'$
$c_1 = 54''$
$c_2 = 16''$

$$\therefore f_2 = 14{,}000 \times \frac{16}{54} \times \frac{17.5}{26} = 2{,}790 \text{ nearly.}$$

For short-span, double-track railway bridges we shall have,

f_1 = 14,000 lbs. per sq. in.
$b = 30'$
$h = 21'$
$c_1 = 84''$
$c_2 = 14''$

$$\therefore f_2 = 14{,}000 \times \frac{14}{84} \times \frac{30}{21} = 3{,}330 \text{ nearly.}$$

For long-span, double-track railway bridges we shall have,

f_1 = 14,000 lbs. per sq. in.
$b = 31'$
$h = 26'$
$c_1 = 84''$
$c_2 = 20''$

$$\therefore f_2 = 14{,}000 \times \frac{20}{84} \times \frac{31}{26} = 4{,}000 \text{ nearly.}$$

Evidently, stresses of this nature are of some importance in ordinary designs, although none of the members but the hangers are hard hit, as a post does not receive its maximum stress when the floor-beam riveted to it is loaded, except in a deck bridge. Whenever wide distances from centre to centre of trusses, shallow floor-beams, or wide posts or hangers are used, the effects should be considered, particularly in the case of hangers. In the Quebec Bridge some of the floor-beams are carried on pins, while those which rivet to the posts are to be so attached that under full load on the floor-beams the said posts will not be bent out of line. In the author's Fratt Bridge, extra section was added to each hanger to provide for this effect.

STRESSES IN A HORIZONTAL PLANE DUE TO LONGITUDINAL DEFORMATION OF CHORDS, ESPECIALLY STRESSES IN FLOOR-BEAMS

As a general rule, the steel floor system of a truss bridge consists of stringers riveted to the webs of the floor-beams. When such a span is loaded, the chords deform longitudinally, while the axial length of the stringers is practically unchanged. If the stringers are riveted continuously from end to end of the span, evidently the floor-beams must bend horizontally, this effect varying from zero at the centre of the truss to a maximum at the ends.

On page 487, Vol. 15, of the *Proceedings* of the American Railway Engineering Association, the assumptions which are usually made for the calculation of these stresses are stated thus:

> "If it is assumed that the axis of the stringers does not elongate, that the stringer connections are unyielding, and that the ends of the beams remain vertically over the joint centres; then it follows that the horizontal deflections of the beams must correspond to the elongations (or shortenings) of the chords. If it is assumed that the centre beam remains straight, the deflection of the adjacent beams must be equal to the elongation (or shortening) of the one panel of the chord; the deflection of the next beam will be equal to the elongation of the two panels, etc."

Suppose we consider any floor-beam at a distance d from a beam which is assumed to remain straight. Then the horizontal deflection Δ of the beam at the point where the stringers are attached to it is, on the above assumptions,

$$\Delta = \frac{sd}{E}, \qquad \text{[Eq. 34]}$$

where s = unit stress in chord,
and E = modulus of elasticity.

Now for a single-track railway bridge with two lines of stringers, it can be shown (see the A. R. E. A. report above referred to) that, assuming the ends of the floor-beam to turn freely, the extreme fibre stress f in the flange of the floor-beam is given by the formula,

$$f = \frac{6\,Ec\,\Delta}{a\,(3b-4a)} = \frac{6\,cd}{a\,(3b-4a)}s, \qquad \text{[Eq. 35]}$$

where c = half width of flange,
b = distance from centre to centre of trusses,
and a = distance from centre of a truss to the nearer stringer.

If we assume as ordinary values

$$b = 17' \, 6'' = 210''$$
$$a = 5' \, 3'' = 63''$$
$$\text{and } s = 13{,}000 \text{ lbs. per sq. in.}$$

we shall have

$$f = \frac{6\,cd}{63\,(630 - 252)} \times 13{,}000 = 3.3\,cd. \qquad \text{[Eq. 36]}$$

In a two-hundred-foot span we would have the following values for the fibre stresses in the end floor-beams:

for $c = 4.2''$ $\quad f = 3.3 \times 4.2 \times 1200 = 16{,}600$;
for $c = 6.2''$ $\quad f = 3.3 \times 6.2 \times 1200 = 24{,}500$.

It is evident from this that if all of the assumptions above made were realized, these bending effects on the floor-beams would be very serious. For double-track bridges the figured fibre stresses will be even higher than for single-track bridges. It will be found, however, that the actual condition in a well-designed structure is much better than has been above assumed. Through and deck spans will be considered separately.

In a through span the bottom flanges of the stringers will be in tension, and will stretch about as much as the chords themselves, thus reducing very materially the bending of the floor-beams. The top flanges of stringers will shorten, and tend to draw away at each floor-beam, thus putting tension in the rivets connecting the floor-beams to the stringers, and bending the connection angles. In order to relieve this condition, the legs of the connecting angles which rivet to the floor-beam web should be as wide as practicable, with the rivet gauges as far apart as possible, thus making the connections somewhat flexible. But the most certain relief is obtained by the provision of an efficient traction frame in every panel, as is required by the specifications of Chapter LXXVIII. These frames will tend to compel the stringers in each panel to move lengthwise with the chords, thus leaving the floor-beams nearly straight. Additional relief can be secured by riveting the stringers after the span has been swung, as the bending effect from the dead-load stretch of the chords can then be made practically zero.

In a deck span the stringers should be made about one-sixteenth ($\frac{1}{16}$) of an inch shorter for each ten feet of length than the clear distance between floor-beams when the truss is on camber blocks, and they should be riveted to the floor-beams only after the span has been swung. The adoption of wide-legged connecting angles will be of advantage, as they can be bent out against the floor-beam webs when riveted, and will then spring back when the chords shorten. The thrust frames, which are always to be provided in deck as well as in through spans, will then have the same effect as in the case of a through bridge.

For long spans the relief given by the methods above mentioned is insufficient, and it is necessary to break the continuity of the stringers. The specifications of Chapter LXXVIII provide that in spans over two hundred (200) feet long at least one expansion joint is to be used; and that in no span is there to be a length of continuously riveted stringers exceeding two hundred (200) feet.

In addition to the bending on the floor-beams, the deformations of the chords produce other secondary effects, one of the most important of which is that on the laterals. The diagonals of the system along the compression chords will be in compression, and those of the system along the tension chords will be in tension. These induced unit stresses in the laterals may be one-half of the primary stresses in the chords, or even more. This indicates the need for ample connections, designed to develop the strength of the diagonals rather than merely to meet the figured stresses. It also shows the importance of making the lateral system along the compression chords of stiff members, as angles capable of sustaining tension only would become slack, unless they were riveted up with the bridge loaded.

TEMPERATURE STRESSES

It is a well-known fact that practically all substances expand and contract as the temperature rises and falls. It is evident, therefore, that all structures will change in length more or less with such variations. Now if the ends of a span be restrained in any way, there are likely to be developed temperature stresses of more or less importance; but if the said ends are free to move, the structure will be nearly free from any such stresses. Evidently, therefore, so far as temperature stresses are concerned, structures should be designed so as to be free to expand or contract in any direction.

In general, it is possible to provide for this movement. Thus spans resting on masonry have one end free to move longitudinally, unless, perchance, they be very short and have their ends rigidly attached thereto. Trestle towers with feet widely spaced and having horizontal struts near the bottom, when properly designed, have one shoe fixed, one free to move transversely, one free to move longitudinally, and one movable horizontally in any direction.

It frequently happens, though, that the provision for movement is undesirable for some reason or other. In many cases such movement would result in a loss of rigidity, and occasionally in a reduction of strength or efficiency. An expansion joint in a floor nearly always involves a poor strip of pavement and frequently also a rough spot in any track which the structure may carry. Furthermore, expansion joints are usually expensive. For all these reasons, therefore, it is of the utmost importance that no unnecessary provision for expansion be made.

Wherever girders or trusses of any ordinary length rest directly on masonry, it will be impossible to omit such provision; for, if this were done, the change in length of the parts would force the shoe to move and either bend over the anchor bolts or crack the masonry. Wherever girders or trusses are carried on columns, there is a possibility that the flexibility of the latter will permit of a reduction in the number of expansion points.

The standard railway trestle consists of alternate long and short spans, the latter being the tower spans. There is usually an expansion joint at one end of each long span. In this type, therefore, there are practically no temperature stresses. Frequently, however, some of the girders are supported on solitary bents. As the columns thereof have some flexibility, it may be permissible to dispense with some of the expansion joints.

In computing temperature stresses in such members, the movement of the top of the column or other member is first to be figured. This is given by the formula,

$$\Delta = 0.0000065LT, \qquad \text{[Eq. 37]}$$

where Δ = movement,

L = distance to point of no movement,

T = temperature change in degrees Fahrenheit.

If L be taken as 100 feet, and T as 100 degrees Fahr., Δ becomes about 0.8 of an inch. That is, the movement for a 100-foot length and 100 degrees Fahr. change in temperature is 0.8 of an inch. We may, therefore, write,

$$\Delta = 0.8 \times \frac{L}{100} \times \frac{T}{100}, \qquad \text{[Eq. 38]}$$

where Δ is in inches, and L in feet.

For steel structures, the maximum value of T may usually be taken as 75 degrees either way from the normal. In some localities smaller figures may be adopted, 45 degrees being sufficient in the tropics. For concrete structures a variation of 30 degrees above and 50 degrees below the normal temperature is generally considered to be sufficient.

Now consider that a column of length l, moment of inertia I, and half width c has its top deflected by an amount Δ, thus developing a force P at the top, and certain bending stresses in the column. If the column be free at one end and fixed at the other, we have the following well-known equations:

$$P = \frac{3\,EI\,\Delta}{l^3}, \text{ or } \Delta = \frac{Pl^3}{3EI}, \qquad \text{[Eq. 39]}$$

$$\text{and } f = \frac{3\,Ec\,\Delta}{l^2}, \text{ or } \Delta = \frac{f\,l^2}{3\,Ec}; \qquad \text{[Eq. 40]}$$

where f = fibre stress at fixed end.

If the column be fixed at both ends, we have

$$P = \frac{12\,EI\,\Delta}{l^3}, \text{ or } \Delta = \frac{P\,l^3}{12\,EI}, \qquad \text{[Eq. 41]}$$

$$\text{and } f = \frac{6\,Ec\,\Delta}{l^2}, \text{ or } \Delta = \frac{f\,l^2}{6\,Ec}; \qquad \text{[Eq. 42]}$$

where f = fibre stress at either end.

It is evident that the length of structure that can be tied together will depend upon the lengths and widths of the columns, and upon the fixity of the ends thereof. The columns are usually fixed at both ends, hinges being used only when their omission would cause the stresses to run unduly high.

Occasionally transverse bracing cannot be carried down close to the ground in viaduct bents. In this case the bases will be made fixed transversely, and if the columns are far apart, the bending stresses transversely due to temperature variation may be of importance. They can be calculated by the formulæ given above.

In hingeless and two-hinged arches the temperature stresses are always of considerable importance; and if the rise of the arch is small, they may be very high. There is another important effect thereof upon both arches and suspension bridges, viz., that variations of temperature cause such a span to deflect. In the long-span East River bridges such deflections are quite large.

There is still another effect, especially in through bridges, viz., that the sun frequently heats the top chord much more rapidly than the bottom chord, which is protected by the floor. This effect causes an upward deflection for a simple span, and a downward one for a cantilever. It is of importance in a swing bridge, as it lowers the ends of the span when swinging, thus making it necessary to provide greater movement in the end-lifting machinery.

In solid concrete structures expansion sets up compressive stresses, which are of no importance. Contraction, however, sets up tensile stresses which will usually crack the concrete at intervals. To prevent this, expansion joints should be introduced not more than fifty (50) feet apart.

The temperature effects on reinforced concrete are peculiar, on account of its composite nature. The coefficients of expansion for concrete and steel are about equal, being 0.0000065 for steel and 0.000006 for concrete. Hence, if any reinforced concrete structure be free to expand or contract, no stresses of any importance will be developed. If the ends of a structure are restrained, and the temperature rises, both the steel and the concrete will be in compression, and no harm will result; but when a fall of temperature occurs, both the steel and the concrete are in tension, which for a fifty (50) degree fall will far exceed the tensile strength of the simple concrete. The primary tendency will be to open up a few large cracks; but if the reinforcing steel is strong enough to break the concrete at any section before the elastic limit of the metal is reached, it will cause the said concrete to separate in many small cracks rather than in a few large ones. It will be found that this condition will require steel amounting in sectional area to more than one-half of one per cent of that of the concrete. Where steel over this amount is used, which is generally the case, there is no danger of any large crack occurring.

INDETERMINATE STRESSES

By the term "indeterminate stresses" is meant those stresses the exact analysis of which is impossible by the principles of statics, but which require for their solution in any structure the consideration of the elastic properties of the materials which enter into its construction. The causes of indeterminate stresses may be divided into three classes, viz.:

1. Redundant members.
2. Redundant reactions.
3. Redundant bending strength.

As an example of the first class, there may be cited a simply supported, multiple intersection truss. The two-hinged arch may be taken as representative of either the first or the second class. Its indeterminateness may be removed by the omission of one of the chord members, in which case it becomes a three-hinged arch, or by the assumption that the abutments can take vertical reactions only, in which case it becomes a simple truss. A continuous girder may be considered to fall under either Class 2 or Class 3. By the removal of all but two reactions, it becomes a simple beam; while by considering its bending strength at certain sections to be zero, the stresses also become determinate.

It will be impossible in this chapter to give more than a brief discussion on the calculation of indeterminate stresses in general. For a clear treatment of nearly all cases which occur in ordinary practice, the reader is referred to "Modern Framed Structures," Parts I and II; while in Molitor's "Kinetic Theory of Engineering Structures" there will be found a very comprehensive treatment of all phases of the subject. The "secondary stresses" already discussed in this chapter are, of course, indeterminate stresses.

The analysis of stresses in indeterminate structures differs in one important feature from that in structures which can be analyzed by the principles of statics. In the latter type, the stresses from any given loads depend only on the loads and the outlines of the structure; while in the former kind, they depend also upon the materials and the cross-sections of the various members. The analysis of indeterminate structures is, consequently, indirect, as the sizes of its parts or the unit stresses therein must be assumed before the stresses in them can be calculated. It is, therefore, necessary to make a preliminary design by an approximate method, and then figure by an exact method, revising the preliminary sections as required, and finally refiguring by the exact method, if necessary.

In the analysis of indeterminate frameworks, two general methods are used.

1. The removal of a sufficient number of members or reactions to make the structure determinate, and the calculation of the effect of replacing these members or reactions.
2. The principle of least work.

The first of these methods is the one generally employed, as it is usually the less laborious. Theoretically, we may remove any members or reactions which will leave the structure statically determinate and stable; but practically much labor may be saved by a proper choice of the redundants to be removed. In general, they should be so chosen that the remaining members will form a simple truss, rather than such a structure as an arch; and it will usually be better to remove a redundant reaction rather than a redundant member. A thorough discussion on the choice of redundant conditions is to be found on page 202 of "Kinetic Theory of Engineering Structures."

After the choice of the redundant conditions has been made, the solution can be worked out along the lines given in either of the texts above mentioned. The methods shown on pages 148 and 149 of "Modern Framed Structures," Part II, are very valuable labor saving devices, and should be thoroughly understood.

The principle of least work will solve such problems from a different view-point. This principle states that when any indeterminate structure is loaded, the stresses will adjust themselves in such a manner that the internal work of deformation of the structure will be a minimum; from which principle it follows that the partial derivative of the internal work with reference to the stress in any redundant member is zero. While the method by the use of the principle of least work is not generally used as much as the one given first, it is advisable that it be clearly understood. It is treated in both of the texts before quoted, and a good discussion is also to be found in Church's "Mechanics of Internal Work."

In the analysis of structures having redundant bending strength at various sections, four methods are available:

1. The differential equation of the elastic line.
2. The removal of a sufficient number of reactions to make the structure determinate, and the calculation of the effect of replacing these reactions.
3. The cutting of the structure at various points, so that it becomes determinate, and the calculation of the effect of again making the structure continuous at the points where it has been cut.
4. The principle of least work.

The first of the above methods is the one generally applied to the calculation of stresses in continuous beams. The computation may be performed either by the double integration of the differential equation, or by the area-moment method, both of which are explained in Chapter XII on "Deflections." The Theorem of Three Moments, given in all standard text books, is derived by either of these methods, and can be applied to advantage in the solution of many problems. Attention might be called to the fact that if the moments of inertia of the various spans are unlike, the Theorem as usually stated is to be modified by replacing each l in the left-hand member of the equation by the corresponding $\frac{l}{I}$, and each l^2 or l^3 in the right-hand member by the corresponding $\frac{l^2}{I}$ or $\frac{l^3}{I}$. This equation is presented in a very general form in an article by J. P. J. Williams, Esq., C.E., in Vol. LXXVI of the Trans. A. S. C. E., entitled "The Theorem of Three Moments." This paper and the accompanying discussion by Prof. Milo S. Ketchum are valuable not only for the form of the Theorem of Three Moments there given, but also on account of the treatment of the subject of continuous beams in general. An equation which has a wider application than the Theorem of Three Moments, and which may be called the Theorem of Four Moments, is derived in a paper by E. F. Johnson, Esq., C.E., in Vol. LV of the Trans. A. S. C. E., entitled "The Theory of Frameworks with Rectangular Panels, and Its Application to Buildings Which Have to Resist Wind." It differs from the Theorem of Three Moments in that it provides for an external moment at any support, and can, therefore, be applied to the analysis of bents two or more stories in height which have no diagonals to take care of transverse loads, but which must depend on the stiffness of the columns and cross-girders or cross-struts. The Theorem of Three Moments will not apply in this case, as there are three or more stiff members meeting at a point. Another convenient type of equation for such cases is one which expresses the moments at the ends of any one span of a continuous beam in terms of the loads, dimensions, and distortions of that span only. The fundamental equation for the solution of secondary stresses, Equation 1 of this chapter, is of this type, as are also the equations used herein for the calculation of stresses due to eccentricity and to transverse loads on a member. In these equations, the end moments are expressed in terms of the angles made by the end tangents of the beam with the straight line joining the ends thereof. These various equations can frequently be used to advantage in the analysis of stresses in continuous beams in general. In many cases, it will be more convenient to employ instead the angles between the end tangents and the original direction of the beam, taking into account also the movements of the two ends of the beam from their original position. This form of equation was proposed many years ago by Prof. Mohr, and was applied by him to the calculation of the secondary stresses in trusses. Prof. Mohr's equation is given in the before-mentioned article by Kunz in *Engineering News* of October 5, 1911, entitled "Secondary Stresses," and also in Grimm's book, "Secondary Stresses in Bridge Trusses." In a Bulletin of the University of Minnesota, issued in March, 1915, this same equation is developed by G. A. Maney, Esq., C.E., in the form in which it can be most conveniently applied to continuous beams; and its application to the design of frameworks and to the calculation of secondary stresses is also shown. The formulæ for the end moment may be stated thus for any beam 1–2 of length l and moment of inertia I, carrying a uniform load of p per lineal unit, and a concentrated load P distant kl from the left end 1:

$$M_1 = \frac{2C_2 - C_1}{3} + \left[3\left(\frac{\delta_2 - \delta_1}{l}\right) + 2\frac{dy_1}{dx_1} + \frac{dy_2}{dx_2}\right]\frac{2EI}{l}, \quad \text{[Eq. 43]}$$

$$M_2 = \frac{C_2 - 2C_1}{3} + \left[3\left(\frac{\delta_2 - \delta_1}{l}\right) + \frac{dy_1}{dx_1} + 2\frac{dy_2}{dx_2}\right]\frac{2EI}{l}, \quad \text{[Eq. 44]}$$

in which δ_1 = deflection perpendicular to line 1–2 at Point 1,
δ_2 = deflection perpendicular to line 1–2 at Point 2,
$\frac{dy_1}{dx_1}$ = angle between line 1–2 and end tangent at Point 1,
$\frac{dy_2}{dx_2}$ = angle between line 1–2 and end tangent at Point 2,
$C_1 = P(k - k^3)\,l + \frac{pl^2}{4}$,
$C_2 = P(2k - 3k^2 + k^3)\,l + \frac{pl^2}{4}$,
M_1 = moment in beam at left end 1,
M_2 = moment in beam at right end 2.

In the above expressions, M_1 and M_2 are to be considered as positive when tending to rotate the supports or the adjacent beams in a clockwise direction, δ_1 and δ_2 as positive when measured downward, and $\frac{dy_1}{dx_1}$ and $\frac{dy_2}{dx_2}$ as positive when the end tangents have rotated in a counter-clockwise direction. (If the beam should be vertical, it is necessary to assume arbitrarily some direction as upward—say the left side.)

This formula can be applied very easily to continuous beams and frameworks of all kinds, being more general than either the Theorem of Three Moments or the Theorem of Four Moments. It is especially valuable for structures in which there are three or more stiff members meeting at a point.

Fig. 37*u* gives values of $k - k^3$ and $2k - 3k^2 + k^3$.

In Bulletin No. 80 of the Engineering Experiment Station of the University of Illinois there appeared an article entitled "Wind Stresses in

Steel Frames of Office Buildings," by W. M. Wilson, Esq., and G. A. Maney, Esq. It discusses some approximate and exact methods proposed by various writers, and gives exact methods based on equations practically identical with Equations 43 and 44. It also contains diagrams which can be utilized for the purpose of making approximate computations.

The two-hinged arch rib is frequently analyzed by the second method, the horizontal reactions being first assumed to be removed and then the effect of replacing them being figured. The third method is usually adopted for the analysis of the hingeless arch rib, the structure being considered as cut at the crown, and then the effect of again making the rib continuous being computed. The fourth method is not often employed, on account of the amount of work involved. For examples of the application of Methods 2 and 3 the reader is referred to Part II of "Modern Framed Structures."

The use of statically indeterminate simple trusses is no longer countenanced in the best American practice. Their employment involves a certain amount of ambiguity of stresses, and the labor they involve in preparing a correct design is very great. It will further be found almost impossible to make all of the members work at economical unit stresses. Where the rigidity of the various systems by which loads can travel is about the same, this effect is slight; but where one path is considerably less rigid than another, the unit stresses in its members will necessarily be low. If, therefore, the adoption of an indeterminate structure be considered advisable for any reason, every effort should be made to ensure that the various paths which the stresses take are, at least approximately, of equal rigidity. It should be further noted that where this equality of rigidities is secured, the loads can generally be assumed to be divided equally between the various systems, and approximate calculations can be made which will be sufficiently accurate for designing purposes. For instance, the Whipple truss can be analyzed with very little error by assuming each system to carry only the loads which come at the panel points of that system. It should also be pointed out that in structures of this nature the number of panels should be a multiple of four, each of the systems being symmetrical about the centre of the span; otherwise

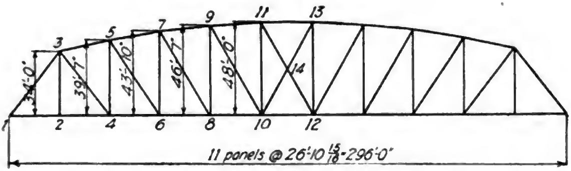


FIG. 11*g*. Truss Diagram of a 296-foot, Single-track-railway, Riveted, Parker-truss Span.

TABLE 11d—CONSTANTS, PRIMARY STRESSES AND ELONGATIONS FOR A 296-

Member	Section	l	A	I	$\frac{I}{l}$	c
1- 3	1 Cov. Pl. $28''\times\frac{5}{8}''$ 2 $\angle^s$ $4''\times4''\times\frac{1}{2}''$ 2 $\angle^s$ $6''\times6''\times\frac{3}{4}''$ 2 Web Pls. $26''\times\frac{3}{4}''$ 2 Web Pls. $26''\times\frac{11}{16}''$	521″	116.63	10,530	20.2	12.5″ top, 14.4″ bottom
3- 5	Cov. Pl. and $\angle^s$ as for 1–3 2 Web Pls. $26''\times\frac{7}{8}''$	330″	87.38	8,830	26.7	
5- 7	Cov. Pl. and $\angle^s$ as for 1–3 2 Web Pls. $26''\times\frac{11}{16}''$ 2 Web Pls. $26''\times\frac{1}{2}''$	327″	103.63	9,790	29.9	
7- 9	Cov. Pl. and $\angle^s$ as for 1–3 4 Web Pls. $26''\times\frac{11}{16}''$	324″	113.38	10,340	31.9	
9–11	Same as for 1–3	323″	116.63	10,530	32.6	
11–13	Same as for 1–3	323″	116.63	10,530	32.6	
1- 2	4 $\angle^s$ $4''\times4''\times\frac{1}{2}''$ 2 Pls. $28''\times\frac{3}{4}''$	323″	57.00	5,280	16.3	14.1″
2- 4	Same as for 1–2	323″	57.00	5,280	16.3	14.1″
4- 6	As for 1–2 2 Pls. $28''\times\frac{9}{16}''$	323″	88.50	7,340	22.7	14.1″
6- 8	As for 4–6 2 Pls. $19\frac{1}{2}''\times\frac{7}{16}''$	323″	105.56	7,880	24.4	14.1″
8–10	As for 1–2 2 Pls. $28''\times\frac{11}{16}''$ 2 Pls. $19\frac{1}{2}''\times\frac{1}{2}''$	323″	115.00	8,420	26.1	14.1″
10–12	As for 1–2 2 Pls. $28''\times\frac{11}{16}''$ 2 Pls. $19\frac{1}{2}''\times\frac{5}{8}''$	323″	119.88	8,580	26.5	14.1″
3- 4	4 $\angle^s$ $4''\times4''\times\frac{3}{4}''$ 2 Pls. $24''\times\frac{3}{4}''$	521″	57.76	4,320	8.3	12.1″
5- 6	4 $\angle^s$ $4''\times4''\times\frac{3}{4}''$ 2 Pls. $20''\times\frac{9}{16}''$	575″	44.26	2,490	4.33	10.1″
7- 8	4 $\angle^s$ $4''\times4''\times\frac{1}{2}''$ 2 Pls. $20''\times\frac{1}{2}''$	617″	35.00	1,890	3.06	10.1″
9–10	4 $\angle^s$ $4''\times4''\times\frac{1}{2}''$ 2 Pls. $18''\times\frac{1}{2}''$	646″	33.00	1,460	2.26	9.1″
11–14	4 $\angle^s$ $6''\times4''\times\frac{1}{2}''$	330″	19.00	180	0.55	6.4″
10–14	4 $\angle^s$ $6''\times4''\times\frac{1}{2}''$	330″	19.00	180	0.55	6.4″
2- 3	4 $\angle^s$ $6''\times4''\times\frac{9}{16}''$	408″	21.24	200	0.49	6.4″
4- 5	4 $\angle^s$ $4''\times4''\times\frac{1}{2}''$ 2 Pls. $20''\times\frac{9}{16}''$	475″	37.50	1,970	4.14	10.1″
6- 7	4 $\angle^s$ $4''\times4''\times\frac{7}{16}''$ 2 Pls. $20''\times\frac{1}{2}''$	526″	33.24	1,760	3.35	10.1″
8- 9	Same as for 6–7	559″	33.24	1,760	3.15	10.1″
10–11	4 $\angle^s$ $3\frac{1}{2}''\times3\frac{1}{2}''\times\frac{3}{8}''$ 2 Pls. $18''\times\frac{7}{16}''$	576″	25.67	1,090	1.89	9.1″

there will be a considerable ambiguity of stress distribution, and, consequently, also an uneconomical use of metal in certain members.

An ordinary swing bridge is statically indeterminate when closed. This is unavoidable. The approximate methods of reaction computation which are generally in use for such structures give results that are perfectly safe in every way and not unduly wasteful of metal. The exact stresses can

FOOT, SINGLE-TRACK-RAILWAY, RIVETED, PARKER-TRUSS SPAN

$12{,}000 \frac{c}{l}$		PRIMARY STRESSES				Δl
Top	Bottom	Dead	Live	Impact	Total	
288	330	395,000*C*	606,000*C*	224,000*C*	1,225,000*C*	−0.182″
454	522	392,000*C*	593,000*C*	220,000*C*	1,205,000*C*	−0.152″
458	527	461,000*C*	707,000*C*	262,000*C*	1,430,000*C*	−0.150″
463	532	502,000*C*	771,000*C*	286,000*C*	1,559,000*C*	−0.148″
464	534	519,000*C*	798,000*C*	295,000*C*	1 612,000*C*	−0.149″
464	534	519,000*C*	798,000*C*	295,000*C*	1,612,000*C*	−0.149″
524		245,000*T*	375,000*T*	139,000*T*	759,000*T*	+0.143″
524		245,000*T*	375,000*T*	139,000*T*	759,000*T*	+0.143″
524		384,000*T*	580,000*T*	215,000*T*	1,179,000*T*	+0.143″
524		456,000*T*	699,000*T*	259,000*T*	1,414,000*T*	+0.144″
524		500,000*T*	768,000*T*	284,000*T*	1,552,000*T*	+0.145″
524		519,000*T*	798,000*T*	295,000*T*	1,612,000*T*	+0.145″
280		213,000*T*	326,000*T*	121,000*T*	660,000*T*	+0.198″
210		138,000*T*	212,000*T*	79,000*T*	429,000*T*	+0.186″
196		84,000*T*	129,000*T*	48,000*T*	261,000*T*	+0.153″
170		40,000*T*	61,000*T*	23,000*T*	124,000*T*	+0.081″
222		0	0	0	0	0
222		0	0	0	0	0
188		39,000*T*	95,000*T*	35,000*T*	169,000*T*	+0.108″
255		129,000C	161,000*C*	60,000*C*	350,000*C*	−0.148″
230		76,000*C*	81,000*C*	30,000*C*	187,000*C*	−0.098″
217		33,000*C*	14,000*C*	5,000*C*	52,000*C*	−0.029″
190		4,000*T*	41,000*T*	15,000*T*	60,000*T*	+0.045″

be calculated by the methods previously described herein for the computation of stresses in indeterminate frameworks, if desired; but ordinarily this is not worth while.

The hingeless type of rib is occasionally adopted for steel arch bridges, and the two-hinged type very frequently. They are preferred by some engineers to the statically determinate three-hinged arch on account of their superior rigidity. For a full comparison of the relative advantages and disadvantages of the various types of arches, see Chapter XXVI.

Continuous trusses and girders, other than those for swing spans, are rarely used in American practice, on account, very largely, of the effect of a possible yielding of the supports. Their economy is either small or nil when due provision is made for uncertainties; and the labor required for their design is very great.

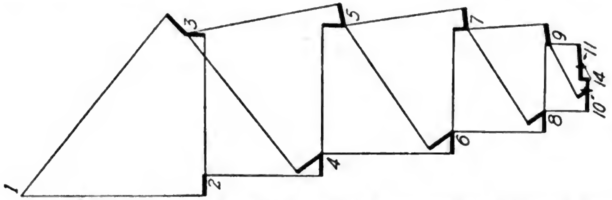


FIG. 11*h*. Williott Diagram for a 296-foot, Single-track-railway, Riveted, Parker-truss Span.

The stiffening trusses of suspension bridges involve indeterminate

TABLE 11*e*

VALUES OF θl, θ, β, AND ϕ FOR A 296-FOOT, SINGLE-TRACK-RAILWAY, RIVETED, PARKER-TRUSS SPAN

Member	θl	θ	β		$\phi = \beta - \theta$			
			Point	Value	Points 1, 4, 8, 12	Points 2, 6, 10	Points 3, 7, 11	Points 5, 9, 13, 14
1- 3..........	1.51	−29.0	1	−33.0	−4.0	.....	−0.8	......
3- 5..........	1.01	−30.6	3	−29.8	.....	.....	+0.8	+ 3.2
5- 7..........	0.79	−24.2	5	−27.4	.....	.....	+4.1	− 3.2
7- 9..........	0.52	−16.1	7	−20.1	.....	.....	−4.0	+ 4.8
9-11..........	0.21	− 6.5	9	−11.3	.....	.....	+3.3	− 4.8
11-13..........	0.00	0.0	11	− 3.2	.....	.....	−3.2	......
13-11..........	0.00	0.0	13	+ 3.2	.....	.....	.....	+ 3.2
1- 2..........	1.19	−36.9	..	......	+3.9	+6.5	.....	......
2- 4..........	0.77	−23.8	2	−30.4	−1.2	−6.6	.....	......
4- 6..........	0.85	−26.3	4	−25.0	+1.3	+3.9	.....	......
6- 8..........	0.60	−18.6	6	−22.4	+4.8	−3.8	.....	......
8-10..........	0.29	− 9.0	8	−13.8	−4.8	+4.5	.....	......
10-12..........	0.00	0.0	10	− 4.5	.....	−4.5	.....	......
12-10..........	0.00	0.0	12	+ 4.5	+4.5	.....	.....	......
3- 4..........	1.16	−22.3	..	......	−2.7	.....	−7.5	......
5- 6..........	0.98	−17.0	..	......	.....	−5.4	.....	−10.4
7- 8..........	0.72	−11.7	..	......	−2.1	.....	−8.4	......
9-10..........	0.38	− 5.9	..	......	.....	+1.4	.....	− 5.4
11-14..........	0.10	− 3.0	14	0.0	.....	.....	−0.2	+ 3.0
10-14..........	0.08	− 2.4	..	......	.....	−2.1	.....	+ 2.4
2- 3..........	0.92	−22.5	..	......	.....	−7.9	−7.3	......
4- 5..........	0.84	−17.7	..	......	−7.3	.....	.....	− 9.7
6- 7..........	0.67	−12.7	..	......	.....	−9.7	−7.4	......
8- 9..........	0.43	− 7.7	..	......	−6.1	.....	.....	− 3.6
10-11..........	0.15	− 2.6	..	......	.....	−1.9	−0.6	......

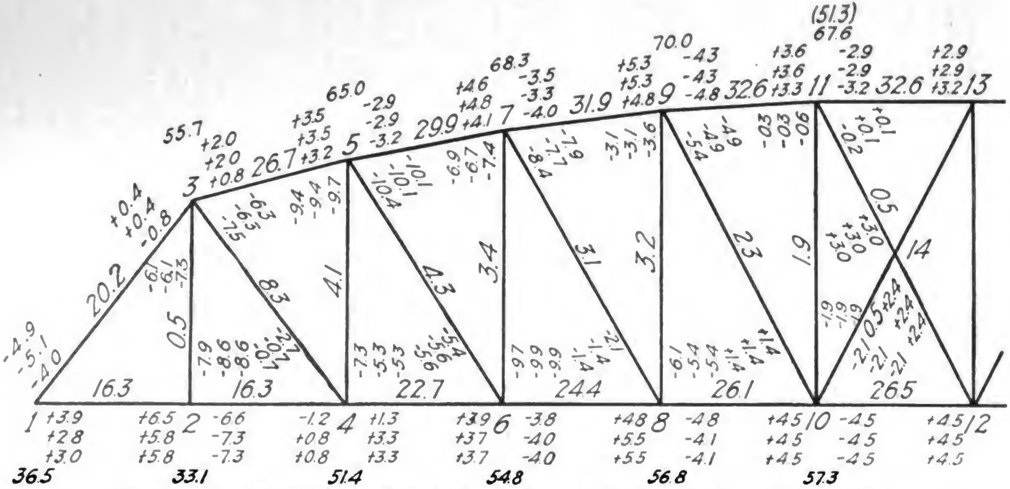


FIG. 11*i*. Trial Values of *T*'s for a 296-foot, Single-track-railway, Riveted, Parker-truss Span.

TABLE 11*f*

SUMMATIONS OF MOMENTS AND TRIAL VALUES OF *T*'S FOR A 296-FOOT, SINGLE-TRACK-RAILWAY, RIVETED, PARKER-TRUSS SPAN

Point	FIRST TRIAL VALUES			SECOND TRIAL VALUES			THIRD TRIAL VAL.	
	$T_{nm} + \frac{1}{2} T_{mn}$	$\frac{I}{l}(T_{nm} + \frac{1}{2} T_{mn})$	Correction	$T_{nm} + \frac{1}{2} T_{mn}$	$\frac{I}{l}(T_{nm} + \frac{1}{2} T_{mn})$	Correction	$T_{nm} + \frac{1}{2} T_{mn}$	$\frac{I}{l}(T_{nm} + \frac{1}{2} T_{mn})$
1	+ 7.2	+117	....	+ 5.7	+ 93	...	+ 5.9	+ 96
	− 4.4	− 89	+12	− 4.9	− 99	...	− 4.7	− 95
				......		...	......	
	Σ	+ 28	+40	......	− 6	...	......	+ 1
2	+ 8.5	+138	− 9	+ 7.2	+118	+1	+ 7.3	+119
	− 7.2	−117	+16	− 6.9	−113	...	− 6.9	−113
	−11.6	− 6	0	−11.6	− 6	...	−11.6	− .6
				......			......	
	Σ	+ 15	+22	......	− 1	0	......	0
3	− 2.8	− 57	....	− 2.1	− 42	...	− 2.1	− 42
	+ 2.4	+ 64	.....	+ 3.8	+102	+2	+ 3.8	+102
	− 8.9	− 74	+ 8	− 6.6	− 55	...	− 6.6	− 55
	−11.3	− 6	....	−10.4	− 5	...	−10.4	− 5
				......			......	
	Σ	− 73	−65	......	0	+2	......	0
4	− 4.5	− 73	....	− 2.9	− 47	...	− 2.9	− 47
	+ 3.3	+ 75	....	+ 5.2	+118	...	+ 5.2	+118
	− 6.5	− 54	....	− 3.8	− 32	...	− 3.8	− 32
	−12.1	− 50	....	−10.0	− 41	...	−10.0	− 41
			....	......		...	......	
	Σ	−102	....	......	− 2	...	......	− 2
5	+ 3.6	+ 96	+16	+ 4.5	+120	...	+ 4.5	+120
	− 1.2	− 36	+11	− 0.5	− 15	−3	− 0.6	− 18
	−13.1	− 56	....	−12.9	− 55	...	−12.9	− 55
	−13.4	− 55	+ 4	−12.0	− 49	...	−12.0	− 49
				......			......	
	Σ	− 51	−20	......	+ 1	−2	......	− 2
6	+ 4.6	+105	+23	+ 5.4	+122	...	+ 5.4	+122
	− 1.4	− 34	+ 8	− 1.3	− 32	...	− 1.3	− 32
	−10.6	− 46	+ 1	−10.6	− 45	...	−10.6	− 45
	−13.4	− 45	+ 1	−13.3	− 45	...	−13.4	− 46
				......		...	......	
	Σ	− 20	+13	......	0	...	......	− 1
7	+ 2.5	+ 75	....	+ 3.4	+102	...	+ 3.2	+ 96
	− 1.6	− 51	....	− 0.7	− 22	...	− 0.9	− 29
	− 9.5	− 29	+ 1	− 8.4	− 26	...	− 8.6	− 27
	−12.2	− 42	....	−11.7	− 40	...	−11.8	− 40
				......		...	......	
	Σ	− 47	−46	......	+ 14	...	......	0
8	+ 2.9	+ 71	....	+ 3.5	+ 86	...	+ 3.5	+ 86
	− 2.6	− 68	....	− 1.8	− 48	...	− 1.8	− 48
	− 3.3	− 20	....	− 5.2	− 16	...	− 5.2	− 16
	− 7.9	− 25	....	− 7.0	− 22	...	− 7.0	− 22
			....			...	......	
	Σ	− 42	....	......	0	...	......	0

TABLE 11*f*—*Continued*

Point	First Trial Values			Second Trial Values			Third Trial Val.	
	$T_{nm} + \frac{1}{2} T_{mn}$	$\frac{I}{l}(T_{nm} + \frac{1}{2} T_{mn})$	Correction	$T_{nm} + \frac{1}{2} T_{mn}$	$\frac{I}{l}(T_{nm} + \frac{1}{2} T_{mn})$	Correction	$T_{nm} + \frac{1}{2} T_{mn}$	$\frac{I}{l}(T_{nm} + \frac{1}{2} T_{mn})$
9	+ 2.8	+ 89	+11	+ 3.6	+115	−3	+ 3.5	+112
	− 3.2	−104	....	− 2.5	− 82	...	− 2.5	− 82
	− 4.7	− 11	....	− 4.2	− 10	...	− 4.2	− 10
	− 6.7	− 21	+ 1	− 5.8	− 19	...	− 5.8	− 19
	Σ	− 47	−35	......	+ 4	+1	......	+ 1
10	+ 2.1	+ 55	+ 9	+ 2.5	+ 65	...	+ 2.5	+ 65
	− 2.3	− 61	....	− 2.3	− 61	...	− 2.3	− 61
	− 1.3	− 3	+ 1	− 1.0	− 2	...	− 1.0	− 2
	− 0.9	− 0	....	− 0.9	0	...	− 0.9	0
	− 2.2	− 4	....	− 2.0	− 4	...	− 2.0	− 4
	Σ	− 13	− 3	......	− 2	...	......	− 2
11	+ 0.9	+ 29	+ 8	+ 1.5	+ 49	...	+ 1.5	+ 49
	− 1.6	− 52	....	− 1.5	− 49	...	− 1.5	− 49
	+ 1.3	+ 1	....	+ 1.6	+ 1	...	+ 1.6	+ 1
	− 1.5	− 3	....	− 1.2	− 2	...	− 1.2	− 2
	Σ	− 25	−17	......	− 1	...	......	− 1
14	+ 2.9	+ 1	....	+ 3.0	+ 1	...	+ 3.0	+ 1
	+ 1.4	− 1	....	+ 1.4	+ 1	...	+ 1.4	+ 1
	− 2.9	− 1	....	− 3.0	− 1	...	− 3.0	− 1
	− 1.4	+ 1	....	− 1.4	− 1	...	− 1.4	− 1
	Σ	0	....	......	0	...	......	0

stresses; and this is absolutely unavoidable, as is explained at length in Chapter XXVII.

The use of hingeless arches, continuous girders, and other indeterminate types of structures is very common in reinforced-concrete bridgework. Since this class of construction, generally speaking, is best built monolithic, such continuity is necessary. Methods of designing for various indeterminate forms will be found in Chapter XXXVII.

There are a great many types of steel construction in which the stresses are strictly indeterminate, although they are never considered to be so. For instance, every plate girder with riveted end connections is more or less fixed at the ends, but this continuity is generally disregarded. The figured sections for the girder itself are, consequently, on the safe side; and as for most types of structures experience has shown that the end moments do no harm, there is no reason for making any extended analysis of their effects. Lateral systems are generally made of double intersection, and when stiff members are employed throughout, the stresses in the two systems are usually assumed to be the same; and as their rigidities are equal, the results are very nearly correct. When the diagonals

TABLE

SECONDARY STRESSES IN THE MEMBERS OF A 296-FOOT,

Member	Point	$T_{nm} + \frac{1}{2}T_{mn}$				
		1st Trial	2nd Trial	3rd Trial	First Trial	
					Top	Bottom
1–3	1	− 4.4	− 4.9	− 4.7	1,270C	
	3	− 2.8	− 2.1	− 2.1		920C
3–5	3	+ 2.4	+ 3.8	+ 3.8		1,250C
	5	+ 3.6	+ 4.5	+ 4.5	1,640C	
5–7	5	− 1.2	− 0.5	− 0.6	550C	
	7	+ 2.5	+ 3.4	+ 3.2	1,150C	
7–9	7	− 1.6	− 0.7	− 0.9	740C	
	9	+ 2.8	+ 3.6	+ 3.5	1,300C	
9–11	9	− 3.2	− 2.5	− 2.5	1,490C	
	11	+ 0.9	+ 1.5	+ 1.5	420C	
11–13	11	− 1.6	− 1.5	− 1.5	750C	
	13	+ 1.6	+ 1.5	+ 1.5	750C	
1–2	1	+ 7.2	+ 5.7	+ 5.9	3,780T	
	2	+ 8.5	+ 7.2	+ 7.3		4,460T
2–4	2	− 7.2	− 6.9	− 6.9		3,780T
	4	− 4.5	− 2.9	− 2.9	2,360T	
4–6	4	+ 3.3	+ 5.2	+ 5.2	1,730T	
	6	+ 4.6	+ 5.4	+ 5.4		2,420T
6–8	6	− 1.4	− 1.3	− 1.3		740T
	8	+ 2.9	+ 3.5	+ 3.5		1,520T
8–10	8	− 2.6	− 1.8	− 1.8		1,360T
	10	+ 2.1	+ 2.5	+ 2.5		1,100T
10–12	10	− 2.3	− 2.3	− 2.3		1,210T
	12	+ 2.3	+ 2.3	+ 2.3		1,210T
3–4	3	− 8.9	− 6.6	− 6.6		2,490T
	4	− 6.5	− 3.8	− 3.8	1,820T	
5–6	5	−13.1	−12.9	−12.9		2,750T
	6	−10.6	−10.6	−10.6	2,230T	
7–8	7	− 9.5	− 8.4	− 8.6		1,860T
	8	− 6.3	− 5.2	− 5.2	1,240T	
9–10	9	− 4.7	− 4.2	− 4.2		800T
	10	− 1.3	− 1.0	− 1.0	220T	
11–14	11	+ 1.3	+ 1.6	+ 1.6	290T	
	14	+ 2.9	+ 3.0	+ 3.0		650T
10–14	10	− 0.9	− 0.9	− 0.9		200T
	14	+ 1.4	+ 1.4	+ 1.4		310T
2–3	2	−11.6	−11.6	−11.6		2,180T
	3	−11.3	−10.4	−10.4	2,130T	
4–5	4	−12.1	−10.0	−10.0	3,090C	
	5	−13.4	−12.0	−12.0		3,420C
6–7	6	−13.4	−13.3	−13.4	3,090C	
	7	−12.2	−11.7	−11.8		2,810C
8–9	8	− 7.9	− 7.0	− 7.0	1,710C	
	9	− 6.7	− 5.8	− 5.8		1,460C
10–11	10	− 2.2	− 2.0	− 2.0	420C	
	11	− 1.5	− 1.2	− 1.2		290C

11*g*

SINGLE-TRACK-RAILWAY, RIVETED, PARKER-TRUSS SPAN

Secondary Unit Stress				Maximum Primary Stress		Second'y Stress in per Cent of Primary
Second Trial		Third Trial				
Top	Bottom	Top	Bottom	Total	Unit	
1,410*C*		1,350*C*		1,225,000*C*	10,500*C*	12.9
	690*C*		690*C*			6.6
	1,990*C*		1,990*C*	1,205,000*C*	13,790*C*	14.4
2,050*C*		2,050*C*				14.9
270*C*		270*C*		1,430,000*C*	13,800*C*	2.0
1,560*C*		1,470*C*				10.7
330*C*		420*C*		1,559,000*C*	13,730*C*	3.1
1,670*C*		1,620*C*				11.8
1,160*C*		1,160*C*		1,612,000*C*	13,830*C*	8.4
700*C*		700*C*				5.1
700*C*		700*C*		1,612,000*C*	13,830*C*	5.1
700*C*		700*C*				5.1
3,000*T*		3,100*T*		759,000*T*	13,330*T*	23.3
	3,780*T*		3,830*T*			28.7
	3,620*T*		3,620*T*	759,000*T*	13,330*T*	27.2
1,520*T*		1,520*T*				11.4
2,730*T*		2,730*T*		1,179,000*T*	13,320*T*	20.5
	2,830*T*		2,830*T*			21.3
	680*T*		680*T*	1,414,000*T*	13,400*T*	5.1
	1,840*T*		1,840*T*			13.7
	950*T*		950*T*	1,552,000*T*	13,490*T*	7.0
	1,310*T*		1,310*T*			9.7
	1,210*T*		1,210*T*	1,612,000*T*	13,440*T*	9.0
	1,210*T*		1,210*T*			9.0
	1,850*T*		1,850*T*	709,000*T*	12,270*T*	15.1
1,070*T*		1,070*T*				8.7
	2,710*T*		2,710*T*	519,000*T*	11,740*T*	23.1
2,230*T*		2,230*T*				19.0
	1,650*T*		1,690*T*	395,000*T*	11,280*T*	15.0
1,020*T*		1,020*T*				9.0
	720*T*		720*T*	305,000*T*	9,240*T*	7.8
170*T*		170*T*				1.8
360*T*		360*T*		225,000*T*	11,840*T*	1.7
	670*T*		670*T*			2.6
	200*T*		200*T*	225,000*T*	11,840*T*	3.0
	310*T*		310*T*			5.7
	2,180*T*		2,180*T*	256,000*T*	12,040*T*	18.1
1,960*T*		1,960*T*				16.3
2,550*C*		2,550*C*		445,000*C*	11,860*C*	21.5
	3,060*C*		3,060*C*			25.8
3,060*C*		3,090*C*		326,000*C*	9,800*C*	31.4
	2,690*C*		2,720*C*			27.8
1,520*C*		1,520*C*		239,000*C*	7,190*C*	21.3
	1,260*C*		1,260*C*			17.6
380*C*		380*C*		165,000*C*	6,430*C*	5.9
	230*C*		230*C*			3.6

are tension members only, all of the stress is assumed to be carried by the system the diagonals of which are in tension. This assumption is, of course, on the safe side. Again, in many cases transverse loads can travel to the ends by two different routes. For instance, a transverse load on the top chord of a through span may pass to the end of the chord through the top laterals, and then down the end posts; or it may go through the transverse bracing down to the bottom chords, and thence through the lower lateral system to the ends of the span. In such cases a rough estimate of the relative rigidities of the different systems must be made to serve as a guide to one's judgment in deciding upon the division of loading to be assumed for the design of the various parts.

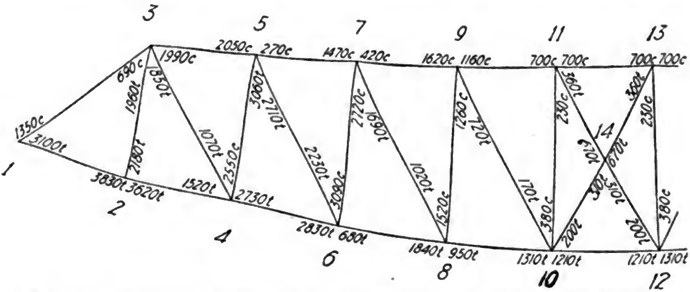


FIG. 11*j*. Secondary-stress Diagram of a 296-foot, Single-track-railway, Riveted, Parker-truss Span.

In conclusion it might be stated that the best way to take care of indeterminate stresses is to avoid them entirely by so designing one's structures as to cut out redundant members and by ever bearing in mind the fundamental principle that "Simplicity is one of the highest attributes of good designing."

CHAPTER XII

DEFLECTIONS

It is very important that the bridge engineer have a clear understanding in regard to the fundamentals of the theory of the deformations in structures, not merely in order to be able to compute deflections, but also because the general principles thereof form the basis for the analysis of statically indeterminate structures of all kinds. It is occasionally necessary to build such structures in steel, and quite frequently in reinforced concrete, although they should be avoided whenever practicable; and furthermore, indeterminate conditions are frequently met with in the design of minor parts of ordinary bridges. If the engineer's knowledge of the basic laws of deflections is superficial, the computation of a problem of such a nature may be quite difficult and frequently unnecessarily tedious; and the results are always likely to be seriously in error. But if he have a clear grasp of the subject, the solution will be comparatively easy and the results dependable.

Most of the important contributions to the theory of deflections have come from European writers, especially German; consequently a thorough study of the question can best be made in the technical literature of that language. The most complete treatment of all phases of the subject that the author has yet seen in any American work is that given by D. A. Molitor, Esq., C.E., in his book entitled "Kinetic Theory of Engineering Structures." The most satisfactory presentation, for the direct use of the busy engineer, is probably that given in "Modern Framed Structures," Parts I and II, by Johnson, Bryan, and Turneaure. While no attempt is made in the latter work to treat fully all of the theory which has been evolved, nevertheless it covers practically everything that is needed in any ordinary problem; and all formulæ are put in a shape well suited for direct use. Hudson's book entitled "Deflections and Statically Indeterminate Stresses" also gives a very good treatment of the entire subject, showing the application of various methods to a number of practical examples. There might also be mentioned several other books which discuss the matter in a manner quite satisfactory. All references made in this chapter to "Modern Framed Structures," Part II, apply to the 1910 edition.

It is beyond the scope of this chapter to enter into any extended discussion of the theory of deflections, the reader being referred concerning it to the text-books above mentioned. However, the practical methods of calculating deflections in beams and trusses will be briefly explained.

Deflections in structures are the result of changes in the lengths of various portions thereof, caused usually by stresses or variations of temperature in the said portions. The engineer is also called upon to compute what are known as inelastic deflections, due to such causes as play in pin-holes, or shop errors in the lengths of members. The deflection of any structure because of certain changes of length in the various portions thereof is the same no matter what the causes which produce these changes may be. Thus, if one member of a truss should elongate by one-quarter of an inch, the resulting deflections of the different panel points are the same, no matter whether the cause of the elongation of the member is a tensile stress, a rise in temperature, play in the pin-holes, the adjustment of a toggle or turn-buckle, or a shop error which resulted in the member being fabricated one-quarter of an inch too long. This important fact should be clearly understood and kept in mind. Formulæ for the deflections of structures will frequently be expressed in terms of stresses or changes of temperature existing therein; but it must be remembered that these quantities appear merely in order to express the changes in length caused thereby.

It is unnecessary to explain how to calculate the change in length of any straight member due to direct stress or a change of temperature. Attention might be called to the fact that a bending moment or transverse shear on such a member will not cause the length of its axis to vary.

The deflection of a straight beam due to bending moment may be calculated by any one of the three following methods:

1. By the double integration of the differential equation of the elastic line.
2. By means of equations derived from the laws of work.
3. By the area-moment method.

The first of these methods is presented in all text-books on the subject, as it is the one first developed, and is the most general in its application. While any extended discussion of it is unnecessary here, it might be well to call attention to the exact meaning of certain of the symbols used therein.

The expression $\frac{M}{EI}$ $\left(\text{which is equal to } \frac{d^2y}{dx^2}\right)$ is the curvature or the *rate* of angular bending of the beam, or the change in the direction of its axis per lineal unit. When multiplied by dx, it expresses the *amount* of angular bending in the length dx; and when the quantity $\frac{Mdx}{EI}$ is integrated for a given length of the beam, the summing of the *amounts* of bending for all of the small lengths dx gives the *amount* of bending in the said length of the beam, which amount is equal to the angle (in radians) between the tangents to the axis of the beam at the extremities of the said length. If the rate of bending is known in terms of any other quantities, it can be inserted in deflection equations in place of $\frac{M}{EI}$. Thus if the unit stresses f_1 and f_2 in the extreme fibers of a beam and its depth d are known, $\frac{M}{EI}$ can be replaced by $\frac{f_1 - f_2}{Ed}$. In this expression, f_1 and f_2 carry their own signs, unit tensile stresses being given one sign, and unit compressive stresses the other. If the values of E for the two faces of the beam be different, as in the case of a reinforced-concrete beam, the corresponding expression would be $\frac{f_1}{E_1 d} - \frac{f_2}{E_2 d}$.

The quantity $\frac{dy}{dx}$, or the slope of the axis of the beam, represents the *rate* at which the deflection of the beam is changing at any given point. When multiplied by dx, it expresses the *amount* of change in the deflection in passing along the length dx; and when the quantity $\frac{dy}{dx}\,dx$ is integrated over a given length of the beam, the summing of the changes in deflection in all of the small lengths dx gives the difference between the deflections of the two points at the extremities of the said length.

In integrating to find the slope or deflection at various points, it is customary to introduce certain "constants of integration," without explaining their meanings. The constant used in performing the first integration to get the value of the slope $\left(\text{or } \frac{dy}{dx}\right)$ at any point is the slope at the point from which the integration begins; and the constant introduced in making the second integration to find the deflection at any point is the deflection at the point where that integration commences. Thus if at a given point in a beam the abscissa be x_1, the slope of the axis be $\frac{dy_1}{dx_1}$, and the deflection be y_1, the slope at any point the abscissa of which is x will be given by the expression,

$$\frac{dy}{dx} = \frac{dy_1}{dx_1} + \int_{x_1}^{x} \frac{Mdx}{EI}, \qquad \text{[Eq. 1]}$$

and the deflection y at this same point by the equation,

$$y = y_1 + \int_{x_1}^{x} \frac{dy_1}{dx_1}\,dx + \int_{x_1}^{x}\int_{x_1}^{x} \frac{Mdx^2}{EI}. \qquad \text{[Eq. 2]}$$

When convenient, the point first mentioned should be taken as the origin of coordinates, so that $x_1 = 0$; and the above equations then become

$$\frac{dy}{dx} = \frac{dy_1}{dx_1} + \int \frac{Mdx}{EI}, \qquad \text{[Eq. 3]}$$

$$\text{and } y = y_1 + \int \frac{dy_1}{dx_1}\,dx + \int\int \frac{Mdx^2}{EI}. \qquad \text{[Eq 4]}$$

The first method is generally much longer than either the second or the third, and should rarely be used when the deflection of but one point of a beam is desired. When the deflections of many points are sought, it may sometimes be the best method to employ.

The second method makes use of the equation,

$$\delta = \int_0^l \frac{Mmdx}{EI}. \qquad \text{[Eq. 5]}$$

In this formula δ is the deflection at *a given point* of the beam; E is the coefficient of elasticity of the material in the beam; and M, m, and I are, respectively, the simultaneous values, at *any* point in the beam, of the moment due to external loads, the moment due to a load of unity acting at the point where the deflection is desired and in the direction in which the deflection is to be found, and the moment of inertia of the beam. The left-hand member expresses the external work done by the unit load in moving a distance δ. In the right-hand member the expression $\frac{Mdx}{EI}$ gives the amount the beam bends in the length dx; and when multiplied by m (the moment produced by the unit load), the result is the internal work performed by the unit load in the portion of the beam dx in length, because the product of a moment into the angle in radians through which it turns is the amount of the work done by the said moment. On summing the products $\frac{Mmdx}{EI}$ for the entire beam, there is obtained the total internal work performed by the unit load in the beam, which is, of course, equal to the external work of the said unit load.

There is no relationship between the quantities M and m, so that we can figure m in any manner we choose, no matter what may be the actual end conditions of the beam. Ordinarily, it will be best to assume the beam simply supported at both ends, unless a cantilever beam is under consideration, in which case it will be simplest to take it as a cantilever. In figuring the deflection at the load point of a beam which is fixed at one or both ends and carries a single concentrated load, it will give the easiest solution to consider the beam to have these same end conditions when computing the values of m. If the beam should be continuous over the supports, and if the values of m should be calculated on this assumption, the right-hand member must either be summed for all of the spans, or else be summed for the one span only and the result increased (algebraically) by the product of the value of m at each end of the span by the angle through which the said end turns. Figuring m as though the beam were simply supported makes its values at the ends thereof zero, so that the labor of finding the work of the end moments is eliminated. If either support moves, the effect of such motion can be figured directly, or else the reaction on the support due to the unit load can be computed and multiplied by the amount of the movement, and the result added (algebraically) to the right-hand member of the equation.

In applying Equation 5, the signs of M and m must be observed, and also the signs of the work performed by the end moments or the end reactions. The product of the angle through which an end of the beam turns into the corresponding m will be positive when m acts in the direction in which the said end rotates; and the product of the reaction of the unit load into the movement of an end support will be positive when the reaction acts in the direction of the said movement. The moment m must represent the action of the beam on the supports or the adjoining spans, not the action of these on the beam; and the reaction employed must be the reaction of the beam on the support, not that of the support on the beam. If the said moment m and the reaction should be taken to represent the action of the supports or of the adjoining spans on the beam, the terms involving them would express external work done on the beam, and would have to be added to the left-hand member of Equation 5.

As has been previously stated, the quantity $\frac{M}{EI}$ in Equation 5 may be replaced by any other expression representing the curvature or rate of bending of the beam.

It will be noted that Equation 5 is analogous both in derivation and application to the well known $\frac{Pul}{AE}$ equation used in computing the deflections of framed structures.

If it should be desired to find the amount of rotation of a given point of a beam, a couple with a moment of unity can be placed at the said point, and Equation 5 applied by changing δ to α (the angle in radians through which the point turns), the values of m being those produced by the unit couple. This new equation merely equates the external work of the unit couple in rotating through the angle α to the internal work performed thereby in the beam.

The application of Equation 5 to deflection problems is well illustrated in Hudson's "Deflections and Statically Indeterminate Stresses"; and the formula is also given in Part I of "Modern Framed Structures." It affords much easier solutions than the first method, and requires about the same amount of work as the third, except when in the latter the results can be written out by inspection.

The third method has been slighted by many American writers, who do not appear to have realized its advantages. It is developed on page 3 of "Modern Framed Structures," Part II; and the principle from which it derives its name is explained on page 6 of the same volume. As most engineers are accustomed to plotting moment diagrams rather than equilibrium polygons, it is convenient to use the term *the area of the moment* *diagram* as meaning the product of the length of the said diagram in feet or inches by the average height thereof in foot-pounds or inch-pounds, rather than the area of the equilibrium polygon. Referring to Fig. 12*a*, the two italicized statements on page 6 of the text last referred to may then be written as follows, for beams with constant moment of inertia:

The angular change between any two points is equal to the area of the corresponding portion of the moment diagram divided by EI.

The deflection of a point B with reference to a tangent at some other point A is equal to the moment about B of the area of the corresponding portion of the moment diagram divided by EI.

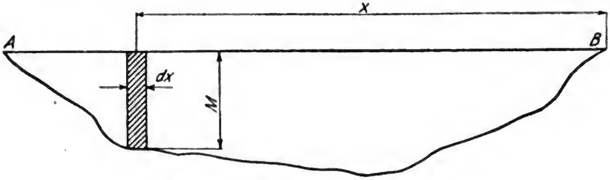


FIG. 12*a*. Area-Moment Diagram.

If the moment of inertia be variable, it will be more convenient to plot as ordinates the values of $\frac{M}{EI}$, rather than the values of M, and then use the above laws in the following form:

The angular change between any two points is equal to the area of the corresponding portion of the $\frac{M}{EI}$ *diagram.*

The deflection of a point B with reference to a tangent at A is equal to the moment about B of the corresponding portion of the $\frac{M}{EI}$ *diagram.*

In some cases it may be advisable to plot as ordinates the values of $\frac{Mx}{EI}$, in which case the deflection can be found by the following rule:

The deflection of a point B with reference to a tangent at A is equal to the area of the corresponding portion of the $\frac{Mx}{EI}$ *diagram.*

The fundamental equations of this method are:

$$\frac{d^2y}{dx^2} = \frac{M}{EI}, \qquad \text{[Eq. 6]}$$

$$\frac{dy}{dx} = \int_A^B \frac{Mdx}{EI}, \qquad \text{[Eq. 7]}$$

and

$$y = \int_A^B \frac{Mxdx}{EI}, \qquad \text{[Eq. 8]}$$

x in the last equation being measured from B as an origin, as shown in Fig. 12a.

The area-moment law can be expressed in a different form from that above presented, so that it will give the deflection of any point B of a beam with reference to the supports, rather than with reference to a tangent at some other point A. It will be noticed that instead of getting the deflection with reference to the tangent at A by means of the laws as already stated, we could have arrived at the same result by conceiving the beam to be a cantilever supported at B and loaded with the area of its $\frac{M}{EI}$ diagram, and then computing the moment at B due to this assumed loading. To get the deflection of the point B of the beam shown in Fig. 12b with reference to the supports, we can in like manner conceive of the beam as being simply supported at the ends and loaded with the area of its $\frac{M}{EI}$ diagram, and then figure the moment at B due to this assumed loading, which gives the deflection at B. The principle of this method may be stated thus:

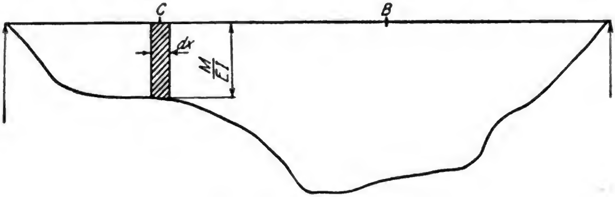


FIG. 12b. $\frac{M}{EI}$ Diagram.

The deflection of any point B of a beam with respect to its supports is equal to the moment which would occur at B if the beam were conceived to be simply supported at the ends and loaded with the area of the $\frac{M}{EI}$ *diagram.*

This law can be proved from Equation 5. In that equation m represents the moment at any point C due to a unit load at B. But in a simply supported beam, m will also equal the moment at B due to a unit load at C, so that $\frac{Mmdx}{EI}$ will represent the moment at B due to a load of $\frac{Mdx}{EI}$ at C. The expression $\int_0^l \frac{Mmdx}{EI}$, therefore, represents the total moment at B due to the loads $\frac{Mdx}{EI}$ on each of the elemental lengths dx throughout the beam; and since the value of this expression is, by Equation 5, equal to the deflection at B, the truth of the law above stated is established.

This second form of the area-moment law is the more general of the two, the first form being a special case thereof. The first form will give the shorter solution when the deflection of but one point is sought, since the second form requires the calculation of the end reactions for the $\frac{M}{EI}$ loading; but the second form will usually be quicker when the deflections of several points are desired.

As has been previously pointed out, the quantity $\frac{M}{EI}$ in any of the statements of the area-moment law can be replaced by any other expression representing the curvature or rate of bending of the beam.

The area-moment method is the simplest of any for problems in either deflections or continuous beams, it being frequently possible to write out the results by inspection. This is well illustrated for deflection problems by the examples shown on pages 7 to 9 of "Modern Framed Structures," Part II; and its value for the solution of problems in continuous girders will be appreciated by comparing the very simple derivation of the theorem of three moments given on pages 16–19 of the text just mentioned with its development by the first method, as expounded in several other works. Another good example of its use is to be found in a paper by E. F. Jonson, Esq., C.E., in Vol. LV of the Trans. A. S. C. E. It can obviously be applied very easily to cases in which the moment of inertia of the beam is not constant. It can also be utilized to great advantage in cases in which exact integration is very difficult or impossible, by plotting to scale either the moment diagram, the $\frac{M}{EI}$ diagram, or the $\frac{Mx}{EI}$ diagram, and summing the areas by a planimeter or in some other manner.

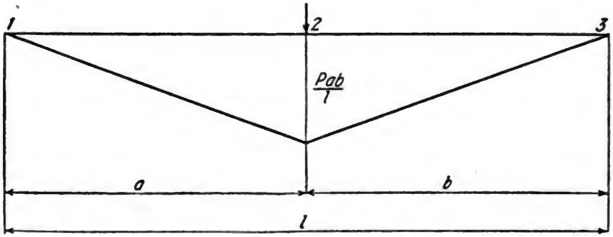


FIG. 12c. Moment Diagram for a Simple Beam with a Concentrated Load.

To illustrate the application of the different methods, let us find the deflection of a simple beam of constant moment of inertia under a concentrated load placed at any given point. See Fig. 12c.

The analysis by the double integration of the differential equation of the elastic curve as is follows:

Considering first the portion 1–2, we have, taking 1 as the origin and 1–2 as the axis of x,

$$EI\frac{d^2y}{dx^2} = M = \frac{Pbx}{l},$$

$$EI\frac{dy}{dx} = EI\frac{dy_1}{dx_1} + \frac{Pbx^2}{2\,l},$$

$$EI\,y = EI\frac{dy_1}{dx_1}\,x + \frac{Pbx^3}{6\,l}.$$

$$\therefore EI\frac{dy_2}{dx_2} = EI\frac{dy_1}{dx_1} + \frac{Pa^2b}{2\,l},$$

and

$$EI\;y_2 = EI\frac{dy_1}{dx_1}\,a + \frac{Pa^3b}{6\,l}.$$

$$\therefore\; EI\frac{dy_1}{dx_1} = \frac{EI\,y_2}{a} - \frac{Pa^2b}{6\,l}.$$

Considering now the portion 2 − 3, we have, taking 2 as the origin and 2 − 3 as the axis of x,

$$EI\frac{d^2y}{dx^2} = M = \frac{Pa}{l}\,(b - x) = \frac{Pab}{l} - \frac{Pax}{l}.$$

$$\therefore EI\frac{dy}{dx} = EI\frac{dy_2}{dx_2} + \frac{Pabx}{l} - \frac{Pax^2}{2\,l}.$$

$$= EI\frac{dy_1}{dx_1} + \frac{Pa^2b}{2\,l} + \frac{Pabx}{l} - \frac{Pax^2}{2\,l}.$$

$$EI\,y = EI\,y_2 + EI\frac{dy_1}{dx_1}\,x + \frac{Pa^2bx}{2\,l} + \frac{Pabx^2}{2\,l} - \frac{Pax^3}{6\,l}.$$

$$\therefore EI\;y_3 = EI\,y_2 + EI\frac{dy_1}{dx_1}\,b + \frac{Pa^2b^2}{2\,l} + \frac{Pab^3}{2\,l} - \frac{Pab^3}{6\,l} = 0.$$

Substituting for $EI\dfrac{dy_1}{dx_1}$ its value as found previously, we have

$$EI\,y_2 + EI\,y_2\left(\frac{b}{a}\right) - \frac{Pa^2b^2}{6\,l} + \frac{Pa^2b^2}{2\,l} + \frac{Pab^3}{3\,l} = 0.$$

$$\therefore EI\,y_2\left(\frac{a+b}{a}\right) = -\frac{Pa^2b^2}{3\,l} - \frac{Pab^3}{3\,l} = -\frac{Pab^2}{3\,l}\,(a+b).$$

$$\therefore y_2 = -\frac{Pa^2b^2}{3\,EI\,l}.$$

The analysis by the equation derived from the laws of work is as follows, using Equation 5.

For the portion 1–2 we have, taking the origin at 1,

$$M = \frac{Pbx}{l},$$

and

$$m = \frac{bx}{l}.$$

For the portion 2–3 we have, assuming the origin at 3,

$$M = \frac{Pax}{l},$$

and

$$m = \frac{ax}{l}.$$

$$\therefore \delta = \int_0^l \frac{Mmdx}{EI} = \frac{1}{EI}\int_0^a \frac{Pb^2\,x^2\,dx}{l_2} + \frac{1}{EI}\int_0^b \frac{Pa^2\,x^2}{l^2}\,dx.$$

$$= \frac{1}{EI}\left[\frac{Pb^2\,x^3}{3\,l^2}\right]_0^a + \frac{1}{EI}\left[\frac{Pa^2\,x^3}{3\,l^2}\right]_0^b$$

$$= \frac{1}{EI}\left(\frac{Pa^3\,b^2}{3\,l^2} + \frac{Pa^2\,b^3}{3\,l^2}\right)$$

$$= \frac{Pa^2\,b^2}{3\,EI\,l}\left(\frac{a+b}{l}\right)$$

$$= \frac{Pa^2\,b^2}{3\,EI\,l}.$$

The analysis by the area-moment method, using the first form, gives directly the deflections of Points 1 and 3 above a tangent through Point 2. Calling these deflections δ_1 and δ_2, we have, since the moment at Point 2 is $\frac{Pab}{l}$,

$$\delta_1 = \frac{Pab}{EI\,l} \times \frac{a}{2} \times \frac{2a}{3} = \frac{Pa^3\,b}{3\,EI\,l},$$

$$\text{and } \delta_3 = \frac{Pab}{EI\,l} \times \frac{b}{2} \times \frac{2\,b}{3} = \frac{Pab^3}{3\,EI\,l}.$$

Now

$$\delta = \delta_1 + (\delta_3 - \delta_1)\frac{a}{l} = \frac{\delta_1(l-a)}{l} + \frac{\delta_3\,a}{l}$$

$$= \frac{\delta_1\,b}{l} + \frac{\delta_3\,a}{l}$$

$$= \frac{Pa^3\,b^2}{3\,EI\,l^2} + \frac{Pa^2\,b^3}{3\,EI\,\;l^2}$$

$$= \frac{Pa^2\,b^2}{3\,EI\,l}\left(\frac{a+b}{l}\right)$$

$$= \frac{Pa^2\,b^2}{3\,EI\,l}.$$

The analysis by the area-moment method, using the second form, gives the deflection δ directly. The reaction R_1, due to the area of the moment diagram, is

$$R_1 = \frac{Pab}{l} \times \frac{b}{2} \times \frac{2\,b}{3\,l} + \frac{Pab}{l} \times \frac{a}{2}\left(b + \frac{a}{3}\right)\frac{1}{l}$$

$$= \frac{Pab}{l^2}\left(\frac{b^2}{3} + \frac{ab}{2} + \frac{a^2}{6}\right) = \frac{Pab}{l^2}\,(a + 2b)\left(\frac{a+b}{6}\right)$$

$$= \frac{Pab}{6\,l}(a + 2b).$$

We then have

$$EI\delta = R_1a - (\text{area } 1\text{–}2) \times \frac{a}{3} = R_1a - \frac{Pab}{l} \times \frac{a}{2} \times \frac{a}{3}$$

$$= \frac{Pa^3b}{6\,l} + \frac{Pa^2\,b^2}{3\,l} - \frac{Pa^3\,b}{6\,l}.$$

$$\therefore \delta = \frac{Pa^2\,b^2}{3\,EI\,l}.$$

The excessive amount of work required by the first method is quite apparent in the preceding problem.

Maxwell's law of reciprocal deflections can frequently be applied to advantage in the calculation of the deflections of beams. A very complete statement of this law is to be found in Molitor's "Kinetic Theory of Engineering Structures" (1910 edition), pages 27–29, but a simpler though less general statement thereof may be made thus:

The displacement in any given direction a′ of any point a of a structure, due to a load P applied at some other point b in a direction b′, is equal to the displacement of the point b in the direction of b′ which would be caused by the application of the load P at the point a in the direction a′.

As an example of the application of Maxwell's law, suppose that we wish to draw the influence line for the deflection of a given point of a beam due to a concentrated load moving across the span. A load of unity is placed at the point in question, and the elastic curve of the beam under this loading is determined by any of the methods previously described and plotted to any desired scale. The curve thus drawn will then be the desired influence line.

The deflections of straight beams due to shear are small as compared with their deflections due to bending moment, and ordinarily they are, in consequence, neglected. They can be computed by means of the formula,

$$y_s = \int_a^b \beta \frac{Qdx}{AE_s}, \qquad \text{[Eq. 9]}$$

in which y_s is the deflection of the point b with respect to a, E_s is the coefficient of elasticity for shear, and Q, A, and β are the values, at any point, of the shear, the area of the beam, and an involved function of the shape of the cross section called by some German writers the "distribution number." If Q, A, and β are constant, Equation 9 takes the form,

$$y_s = \beta \frac{Ql}{AE_s}, \qquad \text{[Eq. 10]}$$

in which l represents the length of the beam from a to b.

The shearing deflection can also be found by an equation similar to Equation 5, of the form,

$$y_s = \int_a^b \beta \frac{Qqdx}{AE_s}, \qquad \text{[Eq. 11]}$$

q being the shear at any point due to a unit load applied at the point the deflection of which is desired. The calculation of deflections by means of Equation 11 is illustrated in Hudson's "Deflections and Statically Indeterminate Stresses."

The quantity β which appears in the above equations cannot be expressed in any simple manner. For rectangular solid beams its value is $\frac{6}{5}$, and for solid circular beams it is $\frac{10}{9}$. For I-beams and girders its value will generally be approximately 2; but it is likely to vary widely therefrom in any particular case. However, since in such beams nearly all the shear is carried by the web, and since the unit shear over the entire web is nearly constant, it will be sufficiently accurate to replace A by A_w, the area of the web, and to make β unity. The error occasioned by the incorrect assumption will never be large; and as the entire deflection due to shear is generally small as compared with that caused by the moment, the approximation is justified. We may, therefore, write for I-beams and plate-girders

$$y_s = \int_a^b \frac{Q dx}{A_w E_s}. \qquad \text{[Eq. 12]}$$

When Q and A_w are constant, this equation becomes

$$y_s = \frac{Ql}{A_w E_s} \qquad \text{[Eq. 13]}$$

In the same manner Equation 11 takes the form,

$$y_s = \int_a^b \frac{Qq dx}{A_w E_s}. \qquad \text{[Eq. 14]}$$

The deflections of curved beams due to moment are found most easily by means of a work equation similar to Equation 5, or by the area-moment method. For good treatments of the subject, the reader is referred to "Modern Framed Structures," Part II, page 112, and to Hudson's "Deflections and Statically Indeterminate Stresses" (1911 edition), page 59. When exact integration is impossible or tedious, it will be most convenient to plot the values of $\frac{M\,ds}{EI}$ $\left(\text{or } \frac{M\,x\,ds}{EI} \text{ or } \frac{M\,y\,ds}{EI}, \text{ if desired}\right)$ at various points along the beam, and sum the areas by the planimeter or in some other approximate manner.

The deflection of a truss due to changes in the lengths of the various members can be computed either analytically or graphically. The analytic method will first be explained.

Suppose that we wish to find by this method the deflection δ of a certain point of a truss in a given direction due to changes of length Δl in the various members. This can be done most easily by placing at the said point a load of unity acting in the given direction, calculating the stresses u in the various members caused by this unit load, and then applying the formula

$$\delta = \Sigma\, u \,.\, \Delta l. \qquad \text{[Eq. 15]}$$

This equation follows directly from the law of the equality of internal and external work, since δ is the external work performed by the unit load in moving a distance δ, the expression $u \,.\, \Delta l$ is the internal work done in one member by the force u moving a distance Δl, and $\Sigma u \,.\, \Delta l$ is the total internal work done in the entire structure.

The changes in length Δl of the various members may be due to stresses, changes of temperature, or such accidental or arbitrary effects as play in pin-holes, shop errors, and adjustments of erecting devices. If any member of length l and area A be subjected to a stress P, a change of temperature of t degrees, and arbitrary or accidental changes of length amounting to $\Delta' l$, the total change in length, Δl, will be given by the formula,

$$\Delta l = \frac{Pl}{AE} + \epsilon t l + \Delta' l, \qquad \text{[Eq. 16]}$$

in which ϵ is the coefficient of expansion of the material. We can then write Equation 15 in the form,

$$\delta = \sum \frac{Pul}{AE} + \Sigma\, u \epsilon t l + \Sigma\, u\, \Delta' l. \qquad \text{[Eq. 17]}$$

This equation is in the form best adapted for direct use.

It is necessary to adopt some convention as to the signs of the various quantities in the above equation. It will be found most convenient to call tensile stresses, rises of temperature, and increases in length positive, and compressive stresses, falls of temperature, and decreases in length negative. A positive value of δ will then indicate that the point moves in the direction in which the unit load was assumed to act, negative that it moves in the opposite direction.

In finding the deflection of a point in one-half of a symmetrical truss under symmetrical loading, it will be best to place a unit load at the corresponding point in the other half also, and then get the value of $\sum \frac{Pul}{AE}$ for one-half of the truss only.

When the deflection of a truss is to be found by the graphical method, either the Williot or the Williot-Mohr diagram is employed. The construction of these diagrams will now be explained.

Suppose that in the truss *ABCFED*, shown in Fig. 12*d*, the members *CF*, *BE*, *AD*, *BC*, and *AB* are shortened by known amounts, while the members *EF*, *DE*, *BF*, and *AE* are lengthened by known amounts; and that the point *F* remains stationary, and the member *CF* fixed in direction. Let the various members be numbered 1, 2, 3, etc., as shown in the figure, and let the change in length of Member 1 be called $\Delta 1$, that of Member 2, Δ2, etc. The deflected positions of the various panel points can now be found as follows:

Since Member 1 shortens an amount Δ1, but does not rotate, the point C will evidently move directly toward F by the amount Δ1, represented by the short heavy line Cc. The point c is, therefore, the deflected position of C. The final position of the point B is next to be found; and for this purpose we must trace the movements of Members 2 and 3. Considering first Member 3, the end B thereof may be deemed to move away from F in a direction parallel to the member itself by an

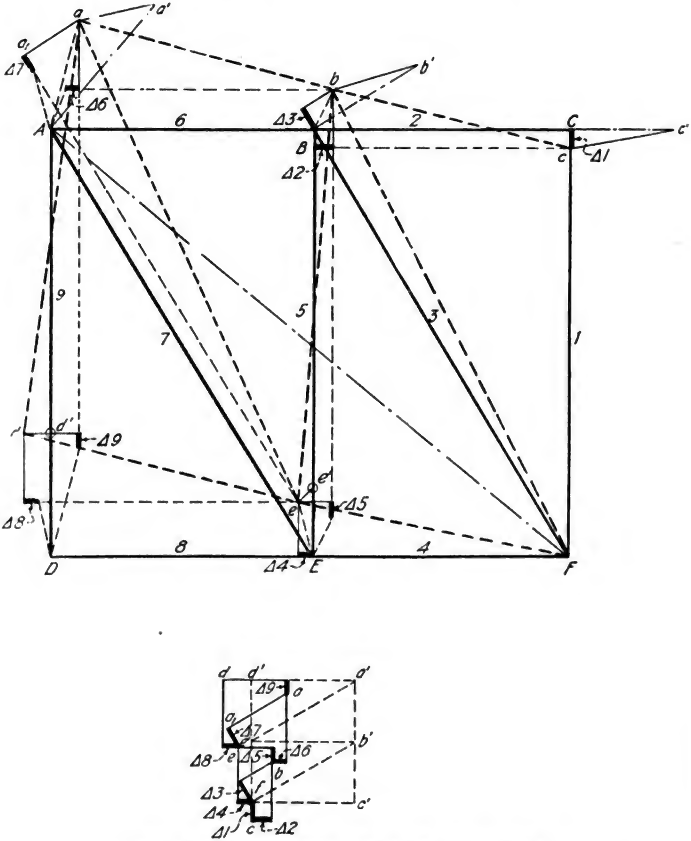


FIG. 12d. Deflection and Williot-Mohr Diagrams.

amount Δ3 represented by the heavy line in the figure, and then the member may be thought of as turning about F as a centre, the end B thus rotating through an arc of radius $BF + \Delta 3$. For Member 2, we may consider that the entire member moves downward an amount Δ1 to the position (parallel to its original position) shown by the light dotted line, then that the end B of the member moves horizontally toward c by an amount Δ2 (shown by the heavy line), and lastly that the member rotates about c as a centre, the end B thus describing an arc of radius $BC - \Delta 2$. The final position of the point B must be at the intersection of the two arcs swung about F and c as centres, which intersection is lettered b in the figure. In an actual truss, these movements will be so small as compared with the lengths of the members themselves that the arcs will be practically straight lines perpendicular to the members. Hence, in making the construction, in the figure, the members themselves have been laid off to a small scale and the lengths Δ1, Δ2, and Δ3 to a much larger one, and the arcs drawn as light straight lines perpendicular to the members. This distortion of scales evidently introduces no errors, since all we desire to know is the position of b with respect to B; and the lines drawn in finding its location are the same in length and direction as they would have been had the members been laid off to the same scale as the distortions themselves. The heavy dotted lines bc and bF do not truly represent the final positions of the members BC and BF; but they give a very good idea of the actual movements of the said members. If the distortions are laid out on a scale fifty times that of the truss itself, the angles between bc and BC and between bF and BF will be approximately fifty times as large as the true values.

The deflected position of Point E is next to be found. Considering first Member 4, the heavy line Δ4 is laid off representing the movement of the end E away from F, and then the light line perpendicular to EF is drawn to indicate its movement of rotation about F as a centre. Next considering Member 5, we can conceive of its moving parallel to itself until the point B reaches b, the point E moving along a line equal in length to the line Bb and parallel thereto. The heavy line Δ5 is then drawn to represent the movement of E toward b due to the shortening of the member, and lastly the light line perpendicular to BE to indicate the rotation about b as a centre. The intersection of these two "arcs" at e gives the final position of E.

We now proceed to find the movement of the point A by the same method that was used for the point E. The member 6 is considered to move parallel to itself to the position shown by the light dotted line, the end B moving along the line Bb and the end A along a line equal and parallel to Bb. The heavy line Δ6 is then drawn to represent the movement of A toward b due to the shortening of the member, and the light vertical line to represent the rotation of A about b as a centre. In like manner, considering Member 7, the point A can be thought of as moving along the light dotted line equal and parallel to *Ee*, then along the heavy line Δ7, and lastly along the light line perpendicular to *AE*. The intersection of the two "arcs" at *a* gives the final position of the point *A*.

Finally, we find the movement of the point *D*. For the member 8, we draw a light dotted line equal and parallel to *Ee*, the heavy line Δ8 representing the movement of *D* away from *e* on account of the elongation of the member, and the light vertical line to indicate its rotation about *e*. For the member 9, we draw the light dotted line equal and parallel to *Aa*, the heavy line Δ9 to show the movement of *D* toward *a* due to the shortening of the member, and, lastly, the light horizontal line to represent its rotation about *a*. The intersection of the two "arcs" at *d* is the deflected position of *D*.

While the method just explained is easy to follow, it makes it necessary to lay out the truss diagram to a fairly large scale, and to draw a good many lines. By an examination of the figure, it will be seen that if construction lines identical with those drawn at all the various panel-points be laid off from a single point, they will unite to form the small compact diagram shown below the larger one. This small diagram can evidently be drawn up much more quickly than the diagrams at the panel points of the large figure; and in order to do this in a logical manner, the work should be carried through as follows:

The point *F*, which is assumed to stand fast, is first located and marked *f*. Since the point *C* merely moves downward toward *F* by an amount Δ1, the heavy line Δ1, drawn downward from *f* locates *c*, the final position of *C*. Since the point *B* moves upward and to the left from *F* by a distance Δ3 parallel to Member 3, and horizontally to the right toward *C* by an amount Δ2, a heavy line of length Δ3 is drawn upward and to the left from *f* parallel to *BF*, and one of length Δ2 horizontally to the right from *c;* and from the ends of these heavy lines are drawn light lines perpendicular to the respective members to represent the rotations of the said members about *F* and *c*. The intersection of these two lines locates *b*, the final position of *B*. Since the point *E* moves horizontally to the left from *F* by an amount Δ4, and vertically upward toward *B* by an amount Δ5, a heavy line of length Δ4 is drawn horizontally to the left from *f*, and one of length Δ5 vertically upward from *b;* and from the ends of these heavy lines are then drawn light lines perpendicular to the respective members representing the rotations of the said members about *F* and *b*. The intersection of these two lines locates *e*, the deflected position of *E*. Since the point *A* moves upward and to the left from *E* a distance Δ7 parallel to the member 7, and horizontally to the right toward *B* by an amount Δ6, a heavy line of length Δ7 is drawn upward and to the left from *e* parallel to *AE*, and one of length Δ6 horizontally to the right from *b;* and from the ends of these heavy lines are drawn light lines perpendicular to the two members, to indicate the rotations thereof about *e* and *b*. The intersection of these two lines at *a* is then the deflected position of the point *A*. As the point *D* moves horizontally to the left from *E* an amount Δ8, and vertically upward toward *A* a distance Δ9, a heavy line of length Δ8 is drawn horizontally to the left from *e*, and one of length Δ9 vertically upward from *a;* and from the extremities of these heavy lines are drawn light lines perpendicular to the two members, to indicate the rotations thereof about *e* and *a*. These two lines intersect at *d*, which is the deflected position of *D*.

By a comparison of the small diagram with the large figure, it will be seen that the positions of the points *c, b, e, a,* and *d* with respect to *f* give directly the deflections of the corresponding panel points with respect to *F*.

The small diagram just discussed was first proposed in 1877 by the French engineer Williot, and is known as the Williot diagram.

After one has familiarized himself with the method just described for the drawing of the Williot diagram, he should be able to construct such figures without difficulty. If he should be somewhat uncertain when starting to draw a diagram, it will be well to lay out a couple of panels free hand, and carry through roughly the constructions at one or two panel-points in the manner first described. After this is done, the method of proceeding with the Williot diagram will be apparent.

It was stated above that the positions of the points *a, b, c,* etc., of the small figure with respect to *f* give the deflections of the points *A*, *B*, *C*, etc., with respect to *F*, or their movements on the assumption that the point *F* does not move. It is further evident that the positions of *a, b, c,* etc., with respect to any other point, as *e*, give the deflections of the points *A*, *B*, *C*, etc., with reference to *E* as a fixed point; so that we are thus able to determine directly from the Williot diagram the deflections of the various panel-points with reference to any one of them, so long as the member which was assumed to remain fixed in direction does not rotate. Also, if it is known that some point has moved a certain distance in the actual structure, a fixed point can be located with reference to this point, and the deflections of all the panel points can then be measured from the said fixed point. Thus, if in Fig. 12*d* it were known that *C* in the actual structure moved 1″ to the left, the member *CF* still remaining fixed in direction, we should locate a reference point 1″ to the right of *c*, and measure all deflections therefrom.

It is frequently impossible to assume as an axis of reference a member which will not rotate in the structure, so that after the Williot diagram has been drawn it may be necessary to determine the effect of rotating the deflected truss through some angle. For instance, suppose that in the problem explained above it were known that the point *D* could move horizontally only. In that case it would be necessary first to draw the Williot diagram as explained previously, and then to determine the effect of rotating the deflected truss until the point *d* drops down to the elevation of the point *D*. It will be found simpler to assume that the original truss rotates about F as a centre until D reaches the elevation of d, rather than that the deflected truss rotates; and as only the relative positions at the various panel points are to be figured, the two methods will evidently give the same results. The simpler method will, therefore, be used. Considering first the large truss diagram, the point D will evidently move vertically to a point d' at the same elevation as d, so that the entire truss will rotate through an angle the value of which in radians is equal to $\frac{Dd'}{DF}$. The point A will move to a point a′, the position of which is found by drawing a line perpendicular to AF, and of a length Aa' given by the formula,

$$Aa' = AF\frac{Dd'}{DF}. \qquad \text{[Eq. 18]}$$

(This line Aa' is really an arc of a circle having its centre at F; but, as explained before, a straight line perpendicular to AF should be used.) The new positions b', c', and e' of the points B, C, and E are then to be found by drawing the lines Bb', Cc', and Ee' perpendicular to the lines BF, CF, and EF, respectively, the lengths of the lines being determined in the same manner as was that of Aa'. Turning now to the small Williot diagram, and remembering that it is but a collection of the construction lines drawn at the various panel-points, it is evident that we can draw thereon lines equal in length and parallel to Dd', Aa', Bb', Cc', and Ee',—all radiating from the point f—and thus locate on this diagram the points d', a', b', c', and e'. It is further clear, however, that these points f, d', a', b', c', and e' constitute a small scale drawing of the truss, with all of its members perpendicular to those in the original truss. Hence we can locate these points most easily by drawing the line fd', assuming it to represent the member FD of the actual truss, and then completing therefrom the small scale drawing of the truss. The positions of the points d, a, b, c, and e with respect to d', a', b', c', and e', respectively, give directly the deflections of the various panel-points in the structure from their original positions D, A, B, etc., on the assumption that F remains fixed in position and D moves horizontally only.

In the example illustrated, the truss was assumed to rotate about the same point that had been used as the starting point in drawing the Williot diagram. It is not essential, however, that these two points be the same. For instance, it might have been known that E was the fixed point and that D remained at the same elevation as E. A line ed' would then have been drawn vertically upward from e (*i. e.*, perpendicular to ED) until it intersected a horizontal through d, and the small-scale truss drawn in on the assumption that ed' represents the member ED of the truss itself. The positions of the points a, b, c, d, and f with reference to the points a', b', c', d', and f', respectively, of this latter truss diagram would indicate the deflections of the various panel-points on the assumptions that E remains fixed and that D moves horizontally only.

Again, it might have been known that *F* moved 1″ downward and 1″ to the right, and that the point *D* moved downward ½″ and was free to move horizontally. We would then have located a point *f′* 1″ to the left of *f* and 1″ above it, and the point *d′* ½″ above *d* and on a vertical line through *f′*. (The last mentioned line is drawn vertical because it must be at right angles to *FD*, which is horizontal.) The small-scale truss would then have been drawn in with *d′f′* representing the member *DF* of the truss, and the positions of the points *a*, *b*, *c*, *d*, *e*, and *f* with reference to the points *a′*, *b′*, *c′*, *d′*, *e′*, and *f′* of the diagram last drawn would represent the movements of the various panel-points.

To give one more illustration, we might suppose that the point *D* remains fixed, and that the point *C* can move horizontally only. We first locate the point *c′* at the intersection of a horizontal line through *c* with a line drawn through *d* perpendicular to a line *DC* drawn in the truss. We then complete the small scale drawing of the truss with points *d* and *c′* representing the points *D* and *C*. As before, the positions of the points *a*, *b*, *c*, *e*, and *f* with reference to the points *a′*, *b′*, *c′*, *e′*, and *f′* of the new diagram give the deflections of the various panel points.

The drawing of the diagram for the effect of rotation was suggested first by Prof. Mohr in 1887; and the complete diagram, showing the effects of both displacement and rotation, is best called the Williot-Mohr diagram.

It will be noted that this diagram does not give directly the positions of the various points with reference to each other, as does the Williot diagram. If this should be desired for the truss shown in Fig. 12*d*, we can proceed as follows. A line is drawn through *a* equal in length and parallel to *a′f*, and the extremity thereof is lettered *a″*; another is drawn through *b* equal in length and parallel to *b′f*, and the extremity thereof is lettered *b″*; etc. The positions of the various points *a″*, *b″*, etc., with reference to *f* give the deflections of the points *A*, *B*, etc., on the assumption that *F* remains fixed and *D* moves horizontally only; and the relative movement of any two points is also shown directly.

If the construction last described gives too complicated a figure, the points can be transferred to a separate diagram at one side. Another method is to take a piece of tracing cloth, mark thereon a reference point, and letter it *f″*. The cloth is then shifted until *f″* coincides with *a′*, and the point *a* is marked thereon and lettered *a″*. This same procedure is then followed through for each of the other points, keeping the cloth properly oriented. The positions of *a″*, *b″*, *c″*, *d″*, and *e″* with reference to *f″* then give the deflections of the corresponding panel-points on the assumptions that *F* remains fixed and that *D* moves horizontally only.

In starting a Williot diagram, a member which will not rotate should be used as a reference axis, if possible. For a symmetrical truss under symmetrical loading, the centre bottom-chord member should be used in the case of a truss with an odd number of panels, and the centre vertical post in the case of a truss with an even number of panels. A symmetrical arch without a crown hinge can also be handled in the same manner. If a truss should have an end fixed in direction, the diagram should be started at that end. For cantilevers, arches with crown hinges and unsymmetrical trusses, it will usually be impossible to select a member which will not rotate.

In drawing up a Williot diagram for a truss with subdivided panels, the positions of the main panel points should first be found, ignoring entirely the secondary members, which do not affect the positions of the said main points. The positions of the various secondary panel-points can then be determined.

The Williot diagram can also be utilized to find the angles through which the various members of a truss have rotated. If we examine the movement of any member, as AE, in Fig. 12d, we find, as before explained, that it may be considered to consist of two parts; first, a movement of translation parallel to itself, and second, a rotation about e as a centre. The angle of rotation γ of this member is evidently

$$\gamma = \frac{a_1 a}{AE}. \qquad \text{[Eq. 19]}$$

Now the line a_1a appears as one of the light lines in the Williot diagram, hence the angle through which any member turns can be figured by means of the light lines in the said diagram. The direction of rotation can be determined in a simple manner. For instance, if we trace from e to a, the rotation is to be considered as one of a about e as a centre, thus giving clockwise rotation. If we trace from a to e, the rotation is to be taken as one of e about a as a center, again giving clockwise rotation. If a member at the centre of a symmetrical truss symmetrically loaded be assumed to remain unchanged in direction, all members in one half of the truss will usually rotate in the same direction.

When a complete study of the deflections of a truss under various loadings is desired, Maxwell's law of reciprocal deflections, which was previously explained, can be employed to advantage. Thus if the influence line for the deflection of any point a of a truss due to a concentrated load applied at the various panel points be desired, a Williot diagram should be drawn for a load at the point a. The deflection of a due to a load at any other point b will then be equal to the deflection at b due to the same load at a, which latter deflection is given by the Williot diagram just constructed. This principle is of especial value in the calculations of stresses in statically indeterminate trusses.

An example of the drawing of a Williot-Mohr diagram for a truss will now be given in full, and the deflection of two points will then be checked by the analytic method. For this purpose the 296′ span used as an example in Chapter XI on Secondary Stresses will be adopted. The calculations for the Williot diagram are shown in Table 11d. The truss is assumed to be fully loaded, and the stresses are figured on this assumption. Care should be taken to see that no error is made in this respect. The stresses given on a stress sheet cannot be employed, as they represent maximum and not simultaneous stresses. Next the changes in length of the various members are figured. The diagram on the right in Fig. 12e is then laid out. It gives the inclinations of all members in the half truss, and is much more compact and convenient than an outline drawing of the truss. The Williot diagram in Fig. 12e is then drawn on the assumption that the centre bottom chord member 10–12 does not rotate, (which is correct) and that Point 10 does not move. The mid-point of the member 10–12 evidently moves to the right by half of the elongation of 10–12 to the point marked *m* in the figure. The mid-point of the member 11–13 evidently lies vertically over the point last located (the truss and its loading being symmetrical), and above it by the amount that the member 10–11 elongates, its position in the diagram being let-

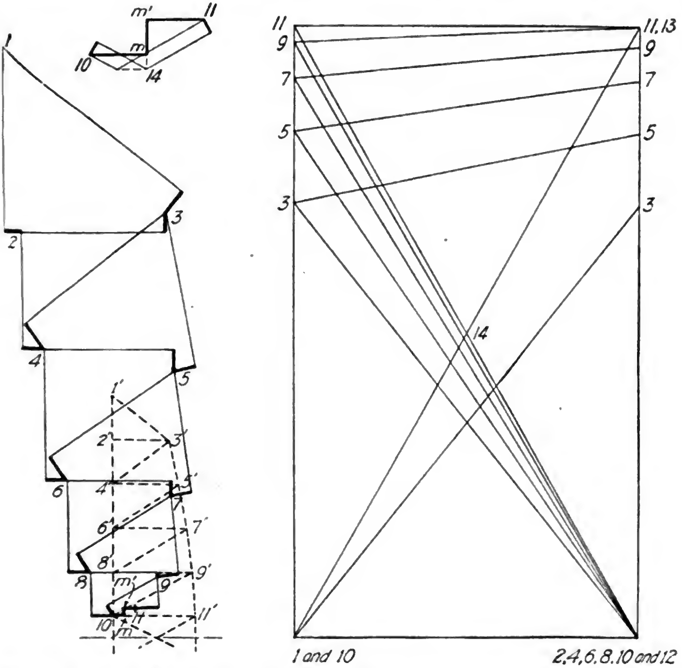


FIG. 12e. Williot-Mohr Diagram for a 296-foot, Single-track-railway, Riveted, Parker-truss Span.

tered m'. Point 11 will evidently move to the right with reference to m' by the amount that half of the member 11–13 shortens. Having found the deflected positions of Points 10 and 11, the constructions to find the deflections of the remaining points should require no explanation, except possibly that for Point 14. The members 10–14 and 11–14 are assumed to have no stress, so that apparently all that is necessary is to draw a line from Point 11 in Fig. 12*e* perpendicular to 11–14, and one from Point 10 perpendicular to 10–14, and take their intersection as the desired point. This point is found to lie to the left of m and m'; but, on account of symmetry, we know that it must lie in the vertical line through these points. Evidently, therefore, Members 11–14 and 10–14 must elongate sufficiently to allow 14 to lie on the said vertical through m and m'. The correct position of Point 14 is on this vertical, and at the same elevation as the intersection just found. The construction lines for locating Point 14 are shown in the large scale sketch just above the main Williot diagram, and are drawn on a scale five times as large. After Point 14 has been located, a line can be drawn therefrom perpendicular to 10–14, meeting a line drawn from Point 10 parallel to 10–14, and another one from Point 14 perpendicular to 11–14, meeting a line drawn from Point 11 parallel to 11–14. These lines give the approximate elongations of 10–14, and 11–14, from which rough values of the unit stresses therein can be computed. For finding an exact result, the stresses in the centre panel would have to be adjusted.

In drawing Fig. 12*e*, it is evident that we could have started with the mid-point of Member 10–12 as the fixed point, and have secured the same diagram as the one shown.

In most cases, the deflections with regard to Point 1 would be the ones desired. These can be determined directly from the Williot diagram.

To illustrate again the use of the Williot-Mohr diagram, one has been drawn in on the assumption that Point 10 remains fixed, and that Member 8–10 does not rotate. No explanation of its construction should be necessary.

The deflection of Point 10 will now be figured by the analytic method, using the device of placing one pound loads both at 10 and 12. The calculations required are shown in Table 12*a*. The quantities $\frac{Pl}{AE}$ are taken directly from the column headed Δl in Table 11*d*, the values for members 10–12 and 11–13 being taken for half of each member. The quantities u are next figured for the one pound loads at Points 10 and 12, and the products $\frac{Pul}{AE}$ are formed. Their algebraic sum is then taken, giving the deflection of either Point 10 or Point 12 below the supports. The result is found to check quite closely with that given by the Williot diagram, as it should.

The calculation of the deflection of Point 8 is also shown in Table 12*a*, to illustrate the fact that the computation of the deflections for additional points is comparatively easy. All stresses u from the end of the truss to Point 8 are the same as before, those in the centre portion only varying. If the deflection of Point 6 were also to be figured, the stresses u in Members 6–7 and 7–8 would change considerably, the stresses in Members 7–9, 9–11, 11–13, 8–10, 10–12, 8–9, 9–10, and 10–11 would be three quarters of those used for Point 8, and all of the other stresses would remain unchanged.

TABLE 12*a*

DEFLECTIONS OF A 296-FOOT, SINGLE-TRACK-RAILWAY, RIVETED, PARKER-TRUSS SPAN

Member	DEFLECTION OF 10			DEFLECTION OF 8	
	$\frac{Pl}{AE}$	u	$\frac{Pul}{AE}$	u	$\frac{Pul}{AE}$
1–3	−0.182	−1.276	+0.232	....	+0.232
1–2	+0.143	+0.791	+0.113	....	+0.113
2–3	+0.108	+0.0	+0.0	....	+0.0
3–5	−0.152	−1.388	+0.211	....	+0.211
2–4	+0.143	+0.791	+0.113	....	+0.113
3–4	+0.198	+0.915	+0.182	....	+0.182
4–5	−0.148	−0.717	+0.106	....	+0.106
5–7	−0.150	−1.864	+0.280	....	+0.280
4–6	+0.143	+1.358	+0.194	....	+0.194
5–6	+0.186	+0.861	+0.160	....	+0.160
6–7	−0.098	−0.713	+0.070	....	+0.070
7–9	−0.148	−2.320	+0.343	....	+0.343
6–8	+0.144	+1.842	+0.265	....	+0.265
7–8	+0.153	+0.890	+0.136	....	+0.136
8–9	−0.029	−0.759	+0.022	+0.241	−0.007
9–11	−0.149	−2.806	+0.418	−2.244	+0.335
8–10	+0.145	+2.308	+0.335	....	+0.335
9–10	+0.081	+0.988	+0.080	−0.135	−0.011
10–11	+0.045	+0.146	+0.007	+0.116	+0.005
½(11–13)	−0.075	−2.800	−0.210	−2.240	+0.168
½(10–12)	+0.072	+2.800	+0.203	+2.240	+0.162
11–14	0.0	0.00	0.0	....	0.0
10–14	0.0	0.0	0.0	....	0.0
			+3.680		+3.392

Either the analytic or the graphic method of figuring deflections can ordinarily be employed for any truss. If the deflection of but one point in one direction only is required, the analytic method is the quicker; but if the movements of several panel-points are to be figured, the graphic method is much shorter. A very common procedure is that of using the graphic method, and then checking the deflection of a single point (generally at the centre) by the analytic method. The latter is particularly useful for determining the movements of a panel-point of a truss which is erected by the cantilever method; for the movement of the said point, due to an adjustment in the length of any given member, can be found directly by multiplying the amount of the adjustment by the value of u for the said member

CHAPTER XIII

COMBINATION OF STRESSES

THE combining of the usual stresses, viz., those due to live load, impact, and dead load (also those for centrifugal load in railroad structures on curves), is simply a matter of addition in order to obtain an equivalent total static load; but when there enter into the combination some of the unusual stresses, such as those due to wind, traction, and temperature, the theory of probabilities has to be employed.

In bridges proper, with the exception of arches having less than three hinges, the only unusual combination is that of the ordinary stresses and the wind stresses, to allow for which standard bridge specifications permit of an increase of thirty (30) per cent over the regular intensities of working stresses; but in trestles, all of the various stresses mentioned may have to be considered, hence the computing of some of the sections for these structures is a complicated matter.

As specified in Chapter LXXVIII, the columns of steel trestles are to be proportioned thus:

First. For live load, impact, centrifugal load, and dead load, with the usual intensities.

Second. For live load, impact, centrifugal load, dead load, and wind load or traction load, with an excess of thirty (30) per cent over the usual intensities.

Third. For live load, impact, centrifugal load, dead load, wind load or traction load, and temperature, with an excess of forty (40) per cent over the usual intensities.

Fourth. For live load, impact, centrifugal load, dead load, traction load, and wind load, with an excess of forty (40) per cent over the usual intensities.

Fifth. For live load, impact, centrifugal load, dead load, traction load, wind load, and temperature, with an excess of fifty (50) per cent over the usual intensities.

The preceding combinations and excess percentages of intensities were adjusted after much deliberation; and this is the first time that such a complete exposition of the matter has been made in print. In the preparation of specifications heretofore it has been deemed too complicated for written treatment and has been left entirely to the judgment of each individual designer. A study of the preceding adjustment will show that the greater the improbability of any combination the greater the intensity of working stress adopted. The worst combination (which, really, never could occur) would stress the metal up to three-quarters of its elastic limit, which is perfectly safe for an occasional loading. It is much better to take into account all possible combinations and to stress the metal high for the worst summation than to ignore such combinations entirely and to trust to luck that they will never occur, as is too generally done in trestle designing. On the other hand, though, it would be extravagant practice to combine all the possible stresses and use either the ordinary intensities or even these increased by the usual thirty (30) per cent allowance for the including of wind. Trestle proportioning hitherto has been rather unscientific; and it is to be hoped that the specifications of Chapter LXXVIII will improve the general practice of designing in this particular at least. When all is said and done, though, it is impracticable to eliminate entirely individual judgment in the designing of high steel trestles, because in some cases local considerations will permit of the reduction or even the ignoring entirely of certain stresses. For instance, when a trestle is situated near the middle of a sharp curve or near the apex of two heavy rising grades, it would be bad judgment to assume a high velocity of train when finding the stresses due to centrifugal loading.

The preceding adjustment of combinations of stresses and intensities of working stresses will apply also to arch structures having less than three hinges per arch, a class of bridge which the author does not recommend for steel structures because of the unavoidable ambiguity of stress that it involves.

The combination of bending stresses and direct stresses is a rather simple matter. In the case of chords of riveted truss railroad bridges subjected to transverse loads there is employed a compromise formula,

$$M = \frac{1}{10} Wl,$$

for finding the bending moment; and the usual intensity must not be exceeded for the combination of extreme fibre bending stress and the direct compression or tension. As the ends of the member are assumed to be fixed, the formula for a load at the middle of the beam would be

$$M = \frac{1}{8} Wl,$$

and that for a load uniformly distributed would be

$$M = \frac{1}{12} Wl;$$

but as the actual conditions lie between these two extremes, the coefficient has been taken as one tenth.

In the case of chords of pin-connected-truss railroad bridges, the ends being assumed free, the compromise formula will be

$$M = \frac{1}{6} Wl,$$

because for a concentrated load at the middle it is

$$M = \frac{1}{4} Wl,$$

and for a uniformly distributed load it is

$$M = \frac{1}{8} Wl.$$

The reason for the compromise is that the ties which rest on the top chords tend to distribute the wheel loads over the total length, but do not do so uniformly. It seems almost unnecessary to say that in the preceding formulæ M is the bending moment, W the total load on the beam, and l its length. Of course, in other cases of loading than that of railroad ties on chords, one of the exact formulæ should be employed instead of the compromise ones.

When the panels are very long and the stiff chords or the web struts are slender, the effect of bending from their own weight is to be considered and combined with the direct stress; but, according to the specifications of Chapter LXXVIII, the intensity of working stress is to be increased ten (10) per cent. Under the assumption of uniformly distributed loading, the formula for bending should be

$$M = \frac{1}{12} Wl$$

for riveted bridges, and

$$M = \frac{1}{8} Wl$$

for pin-connected ones. In the case of struts inclined to the horizontal, the loads thereon are to be resolved into their longitudinal and transverse components, and the latter are to be used in computing the bending moment.

The combination of stresses in cantilever bridges and in arches is discussed in the chapters devoted to those structures, viz., Nos. XXV and XXVI; hence there is no need for further treatment of the matter here, except to call attention to the fact that the sections of members do not require to be increased because of erection stresses, unless such total stresses (including those from wind under an assumed probable pressure of ten (10) or fifteen (15) pounds per square foot) raise the intensities above those specified for a combination of the usual loads with wind.

The combination of stresses in swing spans is treated at length in Chapter LXXVIII, consequently nothing need be said about it here, except to call attention to the fact that where the assumed uplift load stress on any member tends to increase the section it must be considered, but where it tends to diminish the section it must be ignored. The reason for this is that the amount of uplift is assumed arbitrarily, and the probability is that the stresses found by it will never occur.

In summing up stresses care must be used to add only those that can act simultaneously, because some stresses can never occur together; for instance, live load and erection stresses in cantilevers and in arches erected by cantilevering, and live load and wind stresses in highway bridges. This word of warning seems almost unnecessary; nevertheless a careless computer is liable to sum up stresses that cannot act together, as the author knows from personal experience.

In the combination of stresses of opposite kinds distinction is made between the conditions of the reversal. If the cause thereof be wind, the effect of reversion is ignored, because not only is there generally a long interval between reversals but also the maximum wind stress on any piece is of infrequent occurrence. Reversals due to live loads combined with impact are divided into two classes; first, those which occur in succession during the passage of a live load over the structure, and, second, those which are caused by different loadings. In the first case each of the two kinds of stress is to be increased by seventy-five (75) per cent of the other, then the section required for each combination is to be computed and the larger of the two adopted. In the second case the procedure is similar to that just described, except that the percentage to be added is fifty (50) instead of seventy-five (75). In either case, though, when figuring the number of rivets for connecting main members, it is best to add together the two opposite stresses without reducing either and to proportion for the sum, because stress reversal is harder upon the connecting rivets than upon the members themselves which they join. One can appreciate the correctness of this statement by recalling the fact that the quickest way to break off a nail driven for a portion of its length into timber is to bend it a few times in opposite directions—and a rivet is a species of nail.

There arises occasionally in a bridge engineer's practice a question of stress combination that is truly difficult of solution, viz., the testing of an existing structure for the purpose of determining either its safety or its proper carrying capacity, as was the case a few years ago in the Blackwell's Island Bridge at New York City. In the author's opinion, the test loads applied in that case were not the proper ones to use; for a mistake had been made in the designing by omitting the suspended spans in the two long openings and connecting there the meeting ends of the opposite cantilever arms. Instead of finding for each piece the sums of the stresses of like kinds from all of the panel loads throughout the entire structure, the theory of probabilities should have been studied so as to determine for each main member of each span what combinations of live loads affecting it *were likely to occur*, and thus arrive at a common-sense decision concerning the capacity of the bridge to carry moving load. The author understands, though, that the engineers who undertook the job of making the complicated computations for the numerous indeterminate stresses involved were not free agents in regard to this matter, because their task was allotted to them by the "Powers."

This last dissertation brings up another matter that deserves the serious consideration of all practitioners of the specialty of bridge engineering. When an engineer is retained to check the stress sheets and plans of a bridge and to comment thereon, he should always distinguish between structures already built and those not yet constructed, treating the latter far more drastically than the former; because in the latter it is practicable to change anything objectionable that he may discover, while in the former generally it is not. In other words, it is good philosophy for an engineer always to make the best of existing conditions, and not to condemn a structure because it is not what it ought to have been made, but, on the contrary, to show what it is good for under probable and not under practically impossible conditions of traffic.

CHAPTER XIV

INTENSITIES OF WORKING STRESSES

By the term "intensity of working stress" is meant the unit stress or, in American bridge practice, the number of pounds per square inch (either permissible or actual) of net or gross cross-section of a member. Generally the term refers to permissible rather than to actual intensities, especially in specifications for designing. The various intensities of working stresses of which the bridge engineer takes cognizance are those for tension, compression, shear, bending, and bearing. In addition there is the intensity for torsion, which is applicable to bridge designing only in so far as it relates to the machinery of movable spans. There are also intensities for combinations of some of these stresses that act simultaneously; for instance, tension and bending, and torsion and bending; but shear is usually not figured upon as combining with other stresses, although, strictly speaking, it does. Bearing stresses are not combined with any of the others in proportioning the sizes of bridge members. Alternating stresses of tension and compression and reversing stresses in bending are sometimes taken care of by special intensities, but generally they are provided for either by adding the two stresses together and proportioning regularly for the sum, or by adding to each stress a certain percentage of the other, applying the usual intensities to the totals, and adopting the larger of the two results thus obtained. Some years ago certain engineers made a practice of taking into account the maximum and minimum stresses in bridge members by using modifications of the Launhardt formula so as to provide for the destructive effect of oft-repeated stresses. But this custom, which was rather in the nature of a fad, has, with one or two remarkable exceptions, been relegated to oblivion; because no one has yet shown that the repetition of stresses not exceeding the elastic limit has ever produced rupture; and as the actual intensities of working stresses in bridge members rarely exceed one half of that limit, it is evident that in properly designed structures the fatigue of metal is nonexistent and that there is no good reason for any longer considering that mythical bugbear. A trace of it may possibly exist in those modern bridge specifications which provide one intensity of working stress for live loads and another for dead loads; but generally that is done to cover the effect of impact.

The consideration of the latter in bridge designing has altered fundamentally the standard intensities of working stresses and has cut down materially the number of them necessary for specifications, in that by its adoption all live load stresses are supposed to be so augmented that the sum total of stresses on any piece is reduced to an equivalent static load stress. As stated in the chapter on "Impact Loads," the impact method of proportioning bridges is the only scientific, rational, and correct one; and its effect is always to simplify methods of computation, notwithstanding the additional stresses that have to be figured. This method has come to stay. The author has used it exclusively for some twenty years, consequently the specifications for designing given in this treatise are based entirely upon it; and the impact effect is carried into the proportioning of every detail and every rivet affected by the live load, instead of being permitted to govern only the sections of main members, as is the case in certain standard specifications. Theoretically, if the proper amount of impact for all cases be provided, there should be but one working intensity for each kind of stress; but actually this does not prove to be entirely practicable, for two reasons; first, there may be two or three kinds of steel used in any bridge; and, second, there are other considerations than impact which affect the determination of the proper intensities to adopt. For instance, in compression formulæ the author does not consider that it is proper to adopt a single coefficient for $\frac{l}{r}$, irrespective of the condition of the ends of the piece, because a strut with hinged ends is certainly not as strong as a like strut with fixed ends, even when due cognizance is taken of the secondary stresses.

Engineers of railroad companies, in order to anticipate future increase in live loads and other adverse conditions, are somewhat prone to specify unusually low intensities of working stresses; but the author believes that this method is unscientific, and that the future should be provided for by adopting heavy live loads, proper impact, and a large minimum thickness of metal, stressing the steel as high as good practice and experience show to be legitimate. The only sound plea for a reduction of intensities is that it provides for possible deterioration by rust; but even that does not make the method satisfactory, as the amount of provision is proportionate to the total stress, and, therefore, generally to the thickness of the metal, while the rusting is not. If it were advisable to provide for future corrosion, the best way would be to add for every plate and shape a certain amount (say, one-sixteenth ($\frac{1}{16}$) of an inch) to the thickness called for by the computations, taking due cognizance, of course, of the usual requirements for minimum thickness; and the author is inclined to believe that this would be an advisable innovation, although at present he is not prepared to endorse it to the extent of including it in his specifications.

Some engineers contend that intensities of working stresses should be kept low in order to check vibration and reduce impact; but this can be accomplished much more expeditiously and economically by adopting a ballast floor. The shock of passing engines is first felt by the rails and ties, and if these be supported by a mass of ballast, the vibration will be checked materially before it reaches the truss members, and thus the effect of impact will be substantially lessened.

In the days of wrought iron bridges, before impact was considered in designing, it was customary to stress the metal in tension ten thousand (10,000) pounds per square inch, and in compression eight thousand (8,000) pounds per square inch, reduced by Gordon's or Rankine's formula for length over radius of gyration or for length over least diameter; but counters, hip verticals, and flanges of built beams were stressed only eight thousand (8,000) pounds per square inch, and beam hangers six thousand (6,000) pounds per square inch. In compression the usual formula was

$$C = \frac{8{,}000}{1 + \frac{l^2}{ar^2}},$$

where l is the unsupported length of the column in inches, r its radius of gyration in inches, and a a constant depending upon the condition of the ends and often upon the fancy of the engineer doing the computing.

The intensity for shear was generally taken at three-quarters (¾) of that for tension, and that for bearing at twenty-five (25) per cent greater than the same. Some railroad engineers in order to be on the safe side reduced the preceding intensities two thousand (2,000) pounds each, making them about as follows:

Tension..........................	8,000 lbs.
Compression.......................	6,000 lbs. (reduced by formula)
Bending on extreme fibres of beams...	6,000 lbs.
Shear...........................	6,000 lbs.
Bearing on pins....................	10,000 lbs.

In highway bridge designing it was customary to increase by twenty-five (25) per cent the highest intensities used for railroad bridges, making them about as follows:

Tension..........................	12,500 lbs.
Compression.......................	10,000 lbs. (reduced by formula)
Bending on extreme fibres of beams...	10,000 lbs.
Shear...........................	10,000 lbs.
Bearing on pins....................	15,000 lbs.

When steel first came into general use for bridges it was the practice to increase by twenty-five (25) per cent the intensities employed for wrought iron. Some engineers who advocated soft steel stressed it less than that, and some who preferred comparatively high steel stressed it more; but the medium steel which was generally preferred was stressed from ten thousand (10,000) pounds to twelve thousand five hundred (12-500) pounds per square inch in tension and from eight thousand (8,000) pounds to ten thousand (10,000) pounds per square inch in compression (reduced, of course, for $\frac{l}{r}$).

The general adoption of Gordon's formula for struts caused in most cases an extravagant use of metal, at least in comparison with the tension members of the same structure. Another formula often used was Euler's, which was established for large values of $\frac{l}{r}$, and which really does not apply at all to low values thereof. As previously indicated, the fraction l over r appears in these formulæ in the second power. Engineers in their employment followed each other like a flock of sheep till in 1886 Edwin Thacher, the well known bridge engineer, plotted all the reliable records of ultimate strength of wrought iron columns, within practical working limits, upon a diagram in which the abscissas represented the values of $\frac{l}{r}$ and the ordinates the ultimate strengths. He then enclosed the plotted points by a curve, which proved to be approximately an ellipse, through which he drew the major axis and adopted it for the average ultimate strength of wrought iron columns. He made separate plots for various conditions of the column ends, and found in each case that a right line formula gave the best possible average. Then by dividing by the usual factor of safety he obtained formulæ and diagrams for intensities of working stresses.

In 1886 he showed his original plots to the author, and gave him the results of his investigations. Some years later the author applied these formulæ to steel columns by assuming a properly proportionate increase in strength. Still later he modified them so as to provide for the inclusion of impact, and published the results in *De Pontibus*, from which they have been taken for this treatise with certain further modifications in order to be in line with the trend of modern practice, which very properly favors simplification.

One important effect of Thacher's investigation was to inaugurate the custom of stressing short columns much higher than formerly and long ones somewhat less, thus giving metal in compression its proper status and increasing the rigidity of trusses by stiffening the light compression members of the webs. The right-line formula for compression members during the last twenty (20) years has become quite popular; but many specifications still adhere to some modification of Gordon's formula, involving the square of $\frac{l}{r}$. This is due to the preference of certain engineers for using so called rational formulæ, notwithstanding the facts that they are much harder to employ when no tables are at hand, and that it is almost universally conceded that they are no more correct than those of the simpler form. Much has been written on column formulæ, and much valuable gray matter has been expended uselessly on the subject. Probably the most thorough of all the papers ever prepared on the question is that of Mr. J. M. Moncrieff, which was published in the *Transactions* of the American Society of Civil Engineers for 1900. If, however, one will examine those of his diagrams which contain numerous plotted strengths within working limits of $\frac{l}{r}$, and in each thereof will circumscribe these points by a curve, as Thacher did, he will find that the right-line formula will apply very satisfactorily to every case.

Why bridge designers for so many years stressed metal in tension twenty-five per cent higher than in compression on short blocks is difficult to understand; for early experiments showed clearly that the ultimate strength and the elastic limit of both iron and steel are greater in short compression members, where the column effect is practically nil, than they are in tension members. Instead of the intensity for compression being specified lower than for tension, it really should have been made somewhat higher. The Quebec Bridge disaster, which, it is generally acknowledged, was due to the failure of a compression member, has caused some engineers to think that compression members in all bridges have been stressed too high, but such is not the case, as that accident was due to faulty proportioning of section and to entirely inadequate lacing, and not to the abnormally high intensity of working stress for compression, although it is true that the reduction of strength because of $\frac{l}{r}$ was ignored and that the dead load used was far too small, thus making the actual intensities of working stresses improperly high. In the author's opinion, there is no reason whatsoever to fear for the correcness of modern column formulæ, provided that the designing and detailing of these members throughout be properly done, and that the workmanship be truly first-class in every essential particular. Some experiments of his on full-sized columns of both nickel steel and carbon steel prove this. These experiments were discussed at length in *Engineering News* of January 16, 1908; and the complete record of the tests is given in the *Transactions* of the American Society of Civil Engineers for 1909.

If the proper formula for compression members with hinged ends be

$$C = 16{,}000 - 80\frac{l}{r},$$

theoretically that for those with fixed ends should be $C = 16{,}000 - 40\frac{l}{r}$, because in the latter case the length of column between points of contra-flexure is only one-half of that in the former; but practically the effect of some secondary stress must be considered. The amount thereof, though not perfectly determinate, is known to be small; hence the author has assumed as a compromise a coefficient for l over r for fixed ends which is an arithmetical mean between that adopted for hinged ends and the corresponding theoretical one for fixed ends, viz., sixty (60). In writing the specifications of *De Pontibus* he must have done this instinctively for web struts, because he adopted therefor $C = 16{,}000 - 60\,\frac{l}{r}$ for fixed ends and $C = 16{,}000 - 80\,\frac{l}{r}$ for hinged ends, which formulæ he now sees no good object in changing. His reason for lowering the constant of the formulæ for chords and inclined end posts from eighteen thousand (18,000) to sixteen thousand (16,000) is mainly because the medium carbon steel furnished by the rolling mills to-day is somewhat softer than it used to be fifteen or twenty years ago; but he has also been influenced somewhat by the existing general tendency of American bridge specialists to reduce the old compression intensities. For the web members he has made no change in the constant, because subsequent experience has convinced him that the differences in strut intensities between chords and webs, as given in *De Pontibus*, are unnecessary.

As mentioned before incidentally, certain standard specifications, in order to provide for impact and possibly also for other considerations, stress the metal a certain amount for dead load and one-half thereof for live load, irrespective of the length of span or the kind of member. This is a most unscientific method; and a little consideration will show that it is incorrect, because if it is proper for a beam hanger or a hip vertical where the shock is great, it certainly would be incorrect for a bottom chord, upon which the shock is comparatively small. Both theory and experiments have proved that a sudden, or rather instantly-applied, load produces just twice the distortion on any elastic material that the same load does when applied statically; but no member of a bridge receives its greatest load absolutely instantly; hence it is incorrect in proportioning bridge members invariably to stress the metal only one-half as much for live load as for dead load.

If one were to read a number of the bridge specifications that are considered standard in America, he would be struck by the great diversity of intensities of working stresses; for, in truth, no two such specifications agree *in toto* on this question, and many of them differ widely. The principal reasons for this divergence of opinion are as follows:

First. There is quite a difference in the strength of the metal specified, some using soft steel having an ultimate strength of between fifty thousand (50,000) and sixty thousand (60,000) pounds per square inch, others preferring medium steel with an ultimate strength from sixty thousand (60,000) to seventy thousand (70,000) pounds per square inch, and still others striking a mean by adopting metal having an average ultimate strength of sixty thousand (60,000) pounds per square inch, or a trifle more. As explained in Chapter III, the manufacturers, for purely commercial reasons, prefer to furnish such metal, and they influence many engineers to adopt it in their specifications.

Second. The use or the non-use of the impact method of designing makes a great difference in the intensities employed; and, moreover, as engineers have not yet agreed concerning the proper allowances for impact, the intensities they specify when it is included will differ materially, especially as they all appear to aim at practically the same ultimate result, for when the impact allowance is great the intensities are usually taken high, and *vice versa.*

Third. As before stated, some railways continue to specify one intensity for live loads and another much greater for dead loads, and this still further complicates the matter.

Fourth. As previously mentioned, some engineers prefer to anticipate the future by lowering the intensities of working stresses instead of specifying greater loads than those at present in use on railroads.

Fifth. Finally, the personal equation of the specification writer causes variations in intensities, sometimes large but generally small; and this is unavoidable, for there are still many things of importance to learn concerning both the characteristics of the manufactured metal and the action of bridge members under rapidly moving loads.

The intensities of working stresses adopted in the specifications of this treatise for medium and rivet carbon steels, as given in Chapter LXXVIII, are as follows:

Tension on gross sections of eye-bars and reinforcing bars, on net sections of all built members, and on net sections of flanges of all beams	16,000 lbs.
Bending on pins	27,000 lbs.
Bearing on pins	22,000 lbs.
Bearing on shop rivets	20,000 lbs.
Bearing on end stiffeners of plate-girders	16,000 lbs.
Shear on pins	15,000 lbs.
Shear on shop rivets	10,000 lbs.
Shear on plate-girder webs, gross section	10,000 lbs.
Bearing on expansion rollers, in pounds,	600 d,

where d is the diameter of the roller in inches.

For field-rivets the intensities for bearing and shear are to be reduced twenty (20) per cent.

Turned bolts with driving fit are to be stressed the same as field rivets

Compression, in pounds, on struts with fixed ends....$16,000 - 60 \frac{l}{r}$

Compression, in pounds, on struts with hinged ends....$16,000 - 80 \frac{l}{r}$

In these compression formulæ l is the unsupported length of the strut in inches and r is its least radius of gyration in inches.

The intensities for working stresses adopted in the specifications for nickel steel, as given in Chapter LXXVIII, are the following. They have been established on the basis that the least allowable elastic limit in specimen tests is 55,000 lbs. per square inch for plate-and-shape steel and 60,000 pounds for eye-bar steel. In case that a still higher grade of nickel steel is procurable (as it certainly should be), all the intensities, excepting those for rivets, are to be multiplied by the ratio of the higher elastic limit to 55,000 or 60,000, according to the character of the steel under consideration.

Tension on gross sections of eye-bars,	28,000 lbs.
Tension on net sections of all built members, and on net sections of flanges of all beams	26,000 lbs.
Bending on pins	45,000 lbs.
Bearing on pins	35,000 lbs.
Bearing on shop rivets	30,000 lbs.
Bearing on end stiffeners of plate-girders	26,000 lbs.
Shear on pins	23,000 lbs.
Shear on shop rivets	14,000 lbs.
Shear on plate-girder webs, gross section,	16,000 lbs.
Bearing on expansion rollers, in pounds, where d is the diameter of the roller in inches.	$900\ d$,

For field rivets and turned bolts with driving fit the intensities for bearing and shear are to be twenty (20) per cent less than those for shop rivets.

Compression, in pounds, on struts with fixed ends....$26{,}000 - 110\,\frac{l}{r}$

Compression, in pounds, on struts with hinged ends....$26{,}000 - 150\,\frac{l}{r}$

In these compression formulæ, as before, l is the unsupported length of strut in inches and r its least radius of gyration in inches.

The preceding figures are for total equivalent static loads without wind loads added; but when the latter are also included the said figures are to be increased thirty (30) per cent. Members of lateral systems which are subjected to wind loads alone are stressed only as high as truss members for equivalent static loads, excluding wind. Objection has been raised to this on the plea that bridges are seldom, if ever, subjected to the full wind loads specified, and that therefore the intensities for members of lateral systems should be higher—possibly as high as for the combination of all stresses including wind loads. The author has kept down the intensities for wind stresses for the following reasons:

First. As modern lateral systems have rigid intersecting diagonals, there is some ambiguity in the division of loads between them, which ambiguity is never considered.

Second. All the diagonals of modern lateral systems are liable to be subjected to stresses of either tension or compression, and this reversion of stress is universally ignored, because with rare exceptions there are long intervals between reversals of great magnitude.

Third. The attainment of greatest possible stresses in lateral members is much more likely than that of the greatest possible combination of total stresses, including wind, upon truss members.

Fourth. Keeping the intensities fairly low for members of lateral systems increases rigidity by preventing the adoption of very small sections. It is possible that in spans of great length, in which the lateral diagonals become heavy, it would be permissible to stress the larger ones somewhat more than the specifications permit; but this should not be done without giving the question serious consideration.

All of these figures for intensities of working stresses were arrived at after great deliberation and after a study of all the principal bridge specifications in use. They represent the author's convictions as to what is best when everything is considered; and an examination of other specifications shows that the practice of most scientific bridge engineers is in the main closely in accord with them, after due account is taken of the differing allowances for impact.

For the various kinds of timber used ordinarily in bridge construction the intensities of working stress in bending on the extreme fibres are taken as follows, the proper impact being added to the live load stresses:

Long-leaf, Southern yellow pine	2,000 lbs.
Douglas fir or Pacific Coast cedar	1,900 lbs.
White oak	1,800 lbs.
Cypress	1,700 lbs.
Short-leaf yellow pine	1,600 lbs.

These figures differ a little from those given in Chapter LXXVIII, the latter being for two general classes of timber, while the former differentiate between the various kinds.

In addition to the preceding, the specifications give the following intensities of working loads or permissible pressures per square inch on the various substructure materials, when impact is included in the total loads.

Ordinarily good sandstone	200 lbs.
Yellow pine or oak on flat	250 lbs.
Extra good sandstone (not metamorphic)	300 lbs.
Hard brick laid in Portland cement	350 lbs.
Ordinarily good limestone	400 lbs.
Portland cement concrete	500 lbs.
Extra good limestone	550 lbs.
Granitoid	600 lbs.
Metamorphic sandstone of best quality	650 lbs.
Granite	800 lbs.

The greatest allowable intensity of compressive stress in reinforced concrete beams is 600 pounds; and where two compressions at right angles (or inclined) to each other exist simultaneously, their resultant must be found before the intensity is applied. Or, more strictly speaking, after the girder is proportioned on trial the two compressive intensities at right angles (or inclined) to each other are to be figured and their resultant found. It should not exceed the limit just specified; nor, for economy, should it be much less.

The author deems it advisable to give here a statement of his reasons for adopting some of the various preceding intensities and to explain why certain ones differ from others for members of a similar character.

The intensity for bending on pins is much larger than any of the other steel intensities, because experiments have shown that rounds develop very high resistance in bending. Again, the figured bending on a pin is probably seldom attained, because the connected members so adjust themselves under load that the pin-bending is eased, although this can be done only by disturbing the equality of stress distribution on the said connected members and thus stressing some of them unduly high.

The intensity for bearing on pins could probably be raised some two thousand (2,000) pounds without violating the principle adopted in the designing of the celebrated "one-horse shay"; but custom has decreed that the bearing intensity should not be more than forty (40) per cent greater than the tension intensity.

Strictly speaking, the bearing intensity for rivets should bear about the same proportion to that for pins as the ultimate strength of rivet steel does to that of medium steel, or as fifty-five (55) is to sixty-five (65), which would make it about eighteen thousand six hundred (18,600) pounds; but, in reality, the most common ratio of ultimate strengths of rivet steel and medium steel is more nearly equal to $^{55}/_{62}$, which would make the intensity for rivets about nineteen thousand five hundred (19,500) pounds. This agrees closely enough with the twenty thousand (20,000) pounds allowed in the author's specifications.

The permissible shear on pins may appear abnormally high as compared with that on rivets, but it is practically a negligible consideration, because any pin which is large enough for bending and bearing will qualify for shear; besides, the general custom of American bridge engineers of late has been to allow fifteen thousand (15,000) pounds for this intensity. Perhaps the shear on rivets has been made too small. In the first edition of *De Pontibus* the intensity specified was twelve thousand (12,000) pounds; but in the second edition this was changed to ten thousand (10,000) pounds on account of the prevalence of the practice of using very soft steel for rivets. The reduction of twenty (20) per cent in the intensities for field riveting is about right when the rivets are driven by hand, but it is too great for those driven by pneumatic power. However, the author does not believe that the proper time has yet arrived for making a change in this requirement, as there are still a great many field rivets being driven in the old-fashioned way.

The tension allowed on the extreme fibres of timber beams may, at first thought, appear abnormally high; but it must be remembered that this applies mainly to the floor ties of railroad bridges on which the impact allowance is over one hundred (100) per cent. Specifications in which impact is not provided for give this intensity from one thousand (1,000) to twelve hundred (1200) pounds.

The permissible pressures per square inch on the various kinds of masonry have been determined both by the author's experience and by the practice of other engineers. They may appear to some engineers to be high, but it must be remembered that they are to be applied only to total loads which include an allowance for impact. Moreover, they are adjusted for stone and concrete of first-class quality and not for the inferior materials which are too often used in bridgework that is done without proper engineering supervision.

The numerous references in this chapter to the ultimate strength of metal might lead one to suppose that it is the sole criterion adopted in determining intensities of working stresses; but such is by no means the case, for the true criterion is the elastic limit. It was used thus—the intensity for the ordinary combination of stresses, excluding those due to wind, was made about one-half of the minimum elastic limit that is allowed in the specifications, and the combination including wind was permitted to stress the metal thirty (30) per cent higher. But as the elastic limit generally runs some five thousand (5,000) pounds or more per square inch above the minimum allowable, and as the combination of dead load and the greatest specified live, impact, and wind loads is almost an impossibility, it is evident that the metal will rarely, if ever, be stressed higher than one-half of the elastic limit. On the other hand, it must not be forgotten that the elastic limit of the specifications is that of commerce, as determined rather quickly by the drop of the beam, and that the true elastic limit is usually at least two thousand (2,000) pounds per square inch lower; and also that the effects of secondary stresses are usually ignored in bridge designing.

In the case of reversing stresses the author prefers to make arbitrary combinations of the opposite stresses and adhere to the usual intensities rather than to provide separate intensities for the sum of the two stresses. But for those unusual combinations of stresses into which the theory of probabilities enters, as explained in the preceding chapter, he prefers to increase the established intensities by certain percentages.

Just as the first of the manuscript of this book was about to be sent to the printer, the author's attention was called to a description of the Metropolis Bridge over the Ohio River on the line of the Chicago, Burlington, and Quincy Railway, published in *Engineering News* of July 29, 1915, the assumed point of main interest being the use of silicon steel for the built members of the trusses. There is another feature of this design, though, which, in the author's opinion, is of even more importance than the employment of a new alloy of steel, viz., the striking variation from adopted practice in the assumption of live loads and unit stresses. The live load was taken about twenty-eight per cent larger than any hitherto specified in bridge construction; but, on the other hand, the impact allowances are lower than those which are known actually to exist, and the metal is stressed about twenty-five per cent higher than the standard present practice allows. Such befogging of verity in engineering work is to be deplored. Its effect, to say the least, is bewildering and far from conducive to a true valuation of the capacity of the structure. Here can be effectively applied the old proverb, "Two wrongs do not make one right." When one hears that the live load adopted corresponds to what might be called Cooper's Class E 90, he will have altogether too high an opinion of the carrying capacity of the bridge, unless at the same time he is told that the metal is stressed fully twenty-five per cent higher than good practice permits. On the other hand, should one learn that in the design unit stresses up to two-thirds of the elastic limit had been adopted, he would consider the structure unsafe, if he did not know about the enormous assumed live load. The author begs to close this chapter with the emphatic statement that the only proper way to design a bridge to meet future requirements is to assume a live load as great as one thinks can ever come upon the structure, make impact allowances that are in accord with the professional knowledge of the times, acquired by the latest and best experiments, and stress the metal up to one-half of its elastic limit.

CHAPTER XV

FIRST PRINCIPLES OF DESIGNING

Both the student and the practitioner in bridge-designing will do well to recognize and bear constantly in mind certain first principles of design; and to enable them to do so, the author offers the following, which he considers will cover the essential fundamental principles that should govern the designing of all structural metal-work. Most of these will be repeated in the "General Specifications" given in Chapter LXXVIII, under the heading, "General Principles in Designing all Structures," for the reason that the said specifications would be incomplete without them.

The reason for this special chapter being devoted exclusively to these general principles is that the subject is of the utmost importance, and needs much more elaboration than could properly be given it in specifications. On this account the statement of each principle herein will generally be followed by remarks of an explanatory nature giving its *raison d'être* or application. It is to be noticed that the numbering does not agree with that of the "General Principles" in Chapter LXXVIII.

The attention of the reader is called to the fact that this chapter is by far the most important one in the book, in that it contains in a concentrated form the most important conclusions drawn from the author's entire experience in his chosen specialty. The principles given have been established mainly by observation of the mistakes of others, and in a few cases, it must be confessed, by those of his own. Few designers care to make public their errors for fear of the result being to their disadvantage; nevertheless far more is learned from the mistakes of construction than is discovered in any other way. The author would therefore suggest to the reader that he peruse this chapter carefully more than once.

Principle I

Simplicity is One of the Highest Attributes of Good Designing.

It is generally by means of a wide experience only that the young bridge engineer learns the truth of this assertion; but the older he grows and the more knowledge he acquires the more convinced does he become that simplicity, not only in design but also in methods of execution of work, is one of the most important *desiderata*. Other things being equal, that design which is the most simple, or contains the fewest parts, or involves the easiest connections, is the one which will be preferred by competent judges.

PRINCIPLE II

The Easiest Way's the Best.

Although this principle was not enunciated originally in relation to structural work, it nevertheless applies to it just as well, for the most successful engineer is he who in a given time can accomplish in a satisfactory manner the greatest amount of work. This he can do only by the use of every labor-saving device of real value, by systematizing to the greatest practicable extent all that he does, and by making a thorough study of true economy of time and labor.

PRINCIPLE III

The Systemization of All that One Does in Connection with His Professional Work is One of the Most Important Steps that can be Taken Toward the Attainment of Success.

Nor is this by any means all that can be said in favor of establishing a thorough system of doing work; because, in the first place, such a system enables one to accomplish a great deal in a very short time, and, in the second place, it is a subject of the deepest satisfaction and gratification to the man by whom it was evolved.

PRINCIPLE IV

There is an Inherent Sense of Fitness in the Mind of a Well-Trained and Well-Balanced Designer, which Sense of Fitness is of the Greatest Importance in all that He Does.

It is this sense of fitness which enables him often, when inspecting the work of other designers, to see at a glance faults and flaws that would escape the observation of an untrained man. This faculty of rapid and correct judgment is one which can be developed, and one that should receive constant attention throughout an engineer's entire career. It is of special value in an office which employs a large number of draftsmen and computers, all of whose work has to be checked by the head of the office or by a reliable assistant. Nor is it only in connection with the work of others that this faculty is valuable, for it is often serviceable to an engineer on his own personal work, perhaps even without his being conscious thereof, saving him from making errors which pure theory might not enable him to detect, or which the authorities in his line have not yet recognized as errors. An example of this occurred some years ago in the author's practice which will serve to illustrate the point.

In proportioning reinforcing plates at pin-holes, especially for hinged ends, the author had made a practice of instructing his draftsmen to extend these plates considerably beyond the length required by the theoretical number of rivets necessary for the connection, without his being able to give any valid or scientific reason for so doing. By some experiments made later upon the ultimate strength of certain columns with hinged ends, the results of which were published in the *Transactions* of the Engineers' Society of Western Pennsylvania, Thomas H. Johnson showed that such pin-plates, unless extended beyond the length required by the theoretical number of rivets, fail before the full strength of the compression member is developed.

PRINCIPLE V

There are No Bridge Specifications Yet Written, and there Probably Never Will be Any, which will Enable an Engineer to Make a Complete Design for an Important Bridge without Using His Judgment to Settle Many Points which the Specifications Do Not thoroughly Cover; or as Theodore Cooper Puts It: "The most Perfect System of Rules to Insure Success Must be Interpreted upon the Broad Grounds of Professional Intelligence and Common Sense."

At first thought one might conclude that this speaks badly for modern standard bridge specifications, and to a certain limited extent he would be right; for while it is quite true that railway-bridge specifications generally fail to cover the entire ground of ordinary bridge-designing at all adequately, or nearly as thoroughly as they might readily be made to do, nevertheless it is also true that the science of bridge-designing is such a profound and intricate one that it is absolutely impossible in any specification to cover the entire field and to make rules governing the scientific proportioning of all parts of all structures.

The author, however, has done his best in Chapter LXXVIII of this treatise to render the last statement incorrect.

PRINCIPLE VI

In Every Detail of Bridge-Designing the Principles of True Economy Must be Applied by Every One who desires to be a Successful Bridge Engineer.

This subject is such an important one that to its consideration a whole chapter has been devoted.

PRINCIPLE VII

In Bridge-Designing Rigidity is Quite as Important an Element as is Mere Strength.

In fact, each of these properties is dependent upon the other, because if a structure be amply proportioned in its main members for the assumed loads, but improperly sway-braced, the actual dynamic stresses will be greatly in excess of the live-load stresses provided for, and the metal will be overstressed in consequence; while, on the other hand, if rigidity be provided for by ample sway-bracing, but at the same time the main members of the structure be not adequately proportioned, the overstressed metal of the latter will cause vibration to be set up in spite of the sufficiency of sway-bracing. Both of these faults are to be found in existing structures. The effect of the first fault is usually the gradual wearing out of the structure by impact and rack, and that of the second the sudden collapse of the bridge without previous warning.

PRINCIPLE VIII

The Strength of a Structure is Measured by the Strength of its Weakest Part.

This statement is as old as the hills, but is just as valid today as it ever was. The ignoring of its prime importance is constantly the source of waste of metal in structures, fundamentally weak in certain portions, by increasing the weights of other portions, and thus adding to the total load that the weak parts have to carry.

PRINCIPLE IX

In Bridge-Designing Provision Must Always be Made for the Effect of Impact, either by Increasing the Calculated Total Stresses by a Varying Percentage of the Live-Load Stresses, or by Decreasing the Intensities of Working Stresses below those Allowed for Statically Applied Loads.

Different specifications accomplish this result differently, but the former method is undoubtedly the more scientific and rational one.

PRINCIPLE X

In Making the General Layout of any Structure, Due Attention Should be Given to the Architectural Effect, even if the Result be to Increase the Cost Somewhat.

There is no feature of bridge-designing which has been ignored in America to such an extent as has this; and it is only of late years that even a few American engineers have paid any attention whatsoever to æsthetics in that branch of engineering. The subject is such an important one that to its consideration an entire chapter of this book has been specially devoted.

PRINCIPLE XI

For the Sake of Uniformity, and to Conform to the Unwritten Laws of Fitness, It Is Often Necessary in Bridge-Designing to Employ Metal which Is Not Really Needed for either Strength or Rigidity.

The designer who recognizes this fact will usually produce structures of finer appearance than the designer who ignores it because of false notions of economy.

PRINCIPLE XII

Before Starting a Design One Should Obtain Complete Data Therefor.

If he fails to do so, he will generally have to make alteration after alteration as the work progresses; and, as one change usually entails several others, it will result that, by the time the work is finished, enough labor will have been expended thereon to complete two such designs for which proper data were furnished at the outset.

PRINCIPLE XIII

The Building of a Skew-Bridge Should Always Be Avoided when it is Practicable.

It is often possible to square the crossing either by swinging the centre line or by lengthening the span and squaring the piers or abutments. Sometimes, however, it is not practicable to do either, in which case the engineer must make the best of a bad business. The objections to a skew-bridge are these: First, it is fully twice as troublesome to design as a square structure; second, the liability to error in both shop and field is greatly increased by the skew; and, third, the resulting bridge is never so rigid, nor is it so satisfactory in a number of other particulars as a bridge without this objectionable feature.

PRINCIPLE XIV

The Best Modern Practice in Bridge Engineering Does Not Countenance the Building of Structures Having More than a Single System of Cancellation, Except in Lateral Systems where the Resulting Ambiguity of Stress Distribution is of Minor Importance.

Some engineer may question the correctness of this assertion; but if he will glance through the author's paper on "Some Disputed Points in Railway-Bridge Designing," published in the February and March, 1892, number of the *Transactions* of the American Society of Civil Engineers, and reproduced in Mr. Harrington's book entitled "Principal Professional Papers," he will see that, as a whole, the engineering profession endorses the statement.

PRINCIPLE XV

The Employment of a Redundant Member in a Truss or Girder is Never Allowable under Any Circumstances, unless it be in the Mid-Panel of a Span Having an Odd Number of Panels, in Which Case, for the Sake of Appearance, Two Stiff Diagonals Should be Used.

The reason for this is perfectly clear when one considers that it takes extra metal to build the said redundant member, and that its use upsets the calculations of stresses, rendering them in fact insolvable. A lengthy treatise was published some years ago in India upon a method of finding stresses in redundant members, in which much good mental energy was wasted, for the entire book might have been written in these four words: "Never use such members." It is not often that an American engineer is found guilty of employing unnecessary pieces in his designs, but one cannot say the same of his European brethren.

PRINCIPLE XVI

The Use of a Curved Strut or Tie in Bridge-Designing for the Sake of Appearance (or for Any Other Reason) is an Abomination that Cannot for an Instant be Tolerated by a Good Designer.

It is hardly necessary to make such a forcible remark as this to American engineers, although in traveling about the United States one occasionally runs across a violation of the self-evident underlying principle involved in this statement; but the published records of some of the greatest bridges designed by English engineers show the use of pieces of trusses so curved that actually there is compression on one extreme fibre and tension on the other. Architectural effect is undoubtedly a very commendable feature in bridge-designing; but its adoption is no excuse for the violation of the fundamental principle that every compression or tension member of a truss or open-webbed girder should be absolutely straight from end to end. It seems almost unnecessary to state that the appearance of curvature can be obtained by employing short panels and making each chord-length straight between panel points.

PRINCIPLE XVII

In All Structural Metalwork, Excepting only the Machinery for Operating Movable Parts, no Torsion on Any Member Should be Allowed if it Can Possibly be Avoided; Otherwise, the Greatest Care Must be Taken to Provide Ample Strength and Rigidity for Every Portion of the Structure Affected by Such Torsion.

It is not often that this question arises; nevertheless it is sometimes forced upon the consideration of the engineer. It came up once in the author's practice in the case where an elevated-railroad exit-stairway, having at mid-height a landing and a 180-degree turn, had to be supported by a single column in order to comply with the demands of adjacent property owners.

Principle XVIII

The Gravity Axes of All the Main Members of Trusses and Lateral Systems Coming Together at Any Apex of a Truss or Girder Should Intersect in a Point whenever such an Arrangement is Practicable; Otherwise, the Greatest Care Must be Employed to Insure that All the Induced Stresses and Bending Moments Caused by the Eccentricity be Properly Provided For.

This is an important rule that has been more often honored in the breach than in the observance; in fact, it is only lately that American bridge designers have begun to recognize its importance. Even today there are not built many ordinary steel highway bridges in which the desired intersection in a single point of the axes of all members assembling at each apex is accomplished; and in most of these structures where eccentricity exists for want of such intersection, its prejudicial effects are not duly recognized and provided for.

Principle XIX

Truss Members and Portions of Truss Members Should Always be Arranged in Pairs Symmetrically about the Plane of the Truss, Except in the Case of Single Members, the Axes of Which Lie in the Said Plane of Truss.

One occasionally sees a violation of this principle, especially in old bridges; but experience with structures in which it was ignored has been such as to show most clearly that this cannot be done with impunity, for the torsion resulting from eccentrically connected members is patent even to the uninitiated.

Principle XX

In Proportioning Main Members of Bridges, Symmetry of Section about Two Principal Planes at Right Angles To Each Other is a Desideratum to be Attained Whenever Practicable.

Of course, in top-chord and inclined end-post sections, which almost always should be designed with a cover-plate, symmetry about both principal planes is not attainable. The objectionable features caused by want of it, however, are provided against by the next "principle."

Principle XXI

In Both Tension and Compression Members the Centre Line of Applied Stress Must Invariably Coincide with the Axial Right Line Passing through the Centres of Gravity of all Cross-Sections of the Member Taken at Right Angles Thereto.

Until a few years ago this important principle was simply ignored, the effect being that the allowed intensities of working stresses were often exceeded by from fifty to one hundred per cent because of the eccentricity thus involved.

PRINCIPLE XXII

The Principle of Symmetry in Designing Must be Carried Even into the Riveting; and Groups of Rivets Must be Made to Balance about Central Lines and Central Planes to as Great an Extent as is Practicable.

The violation of this principle was exceedingly common not very long ago; and even today, when checking the shop drawings of some of the leading bridge-manufacturing companies, the author's assistants have to correct occasional departures from it.

PRINCIPLE XXIII

Unless there be Some Good Constructive Reason Necessitating the Contrary, Every Compression Member Should be So Designed That the Metal is Kept as Far from the Two Principal Axial Planes as the Rules of Good Proportioning Will Permit, in Order to Make the Radii of Gyration as Large as Practicable. For Economic Reasons the Two Principal Radii of Gyration Should be Made as Nearly Equal to Each Other as the Controlling Conditions Will Allow.

It was a violation of this principle that was the underlying reason for the mistake in design which caused the great Quebec Bridge disaster.

PRINCIPLE XXIV

Every Compression Member Should be So Well Provided with Lacing or Latticing and Stay Plates, that Were the Piece Tested to Destruction it would Fail as a Whole and Not by Reason of the Details. For Chord Sections a Continuous Cover-Plate Should be Used Whenever Practicable, and for Very Large Chord Sections Two Covers Should be Adopted, One above and One Below, Provision being Made to Permit Thorough Interior Painting.

Defective latticing, combined with the violation of Principle XXIII, was the mistake in design referred to in the comment on that "Principle."

PRINCIPLE XXV

In Proportioning Members of Bridges to Meet Stresses and Combinations of Stresses it is Important to Consider Duly the Quality, Frequency, and Probability of the Action of the Said Stresses or Combinations of Stresses.

As a rule, standard specifications take care fairly well of this subject;

nevertheless there will often occur in one's practice cases which they do not cover, for instance, in the designing of high trestles. In such cases the frequency of application of stress should be considered; because, if a certain stress or combination of stresses be of frequent occurrence, the usual intensity of working stress should be adopted, while for very infrequent occurrences the said intensity can, with perfect safety, be taken considerably higher. Again, the probability of the application of a certain load or loads should be considered; because for inevitable loads or combinations of loads the metal should be stressed as is usual, while for highly improbable loads or combinations of loads it is legitimate to stress it much higher.

Principle XXVI

In All Main Members Having an Excess of Section above that Called for by the Greatest Combination of Stresses, the Entire Detailing Should be Proportioned to Correspond with the Utmost Working Capacity of the Member, and Not Merely for the Greatest Total Stress to Which it May be Subjected. In this Connection, though, the Reduced Capacity of Single Angles Connected by One Leg only Must Not be Forgotten.

It is almost needless to state that most engineers, especially those connected with contracting companies, will disagree with the author on the correctness of this statement; nevertheless the latter has yet to see the first case where adherence to the principle would involve improper, clumsy, or inappropriate construction. If it be right, for any reason, to use an extra amount of metal in the section of a member, why is it not also right to design that member throughout so that, if tested to destruction, it would fail as a whole and not in a detail? It seems to the author that the considerations which require extra section would demand either extra strength or extra rigidity, or both, in the details as well as in the section itself.

Principle XXVII

In Every Bridge and Trestle Adequate Provision Must be Made for Contraction and Expansion.

Neglecting to comply with this principle has often been the cause of failure and disaster.

Principle XXVIII

No Matter How Great Its Weight May Be, Every Ordinary Fixed Span Should be Anchored Effectively to its Supports at Each Bearing Thereon.

At one end it should be anchored immovably, and at the other so as to provide for longitudinal expansion and contraction. Such anchorage prevents the dislodging of the structure by wind-pressure or by an accidental blow from a moving object. It also prevents the structure from creeping.

PRINCIPLE XXIX

The Bridge-Designer Should Never Forget that it is Essential throughout Every Design to Provide Adequate Clearance for Packing, and to Leave Ample Room for Assembling Members in Confined Spaces.

There is no more fruitful source of profanity for bridge-erectors than the neglect of this principle; and as nearly every designer has to spend a year or two in learning to allow enough clearance, it follows that the said bridge-erectors should be given the benefit of "extenuating circumstances" when brought to judgment for their notorious addiction to the use of strong language.

PRINCIPLE XXX

Although for Various Reasons Engineers are Agreed that Field-Riveting Should be Reduced to a Minimum, such an Opinion Should not be Allowed to Militate Against the Employment of Rigid Lateral Systems. All Designs Should be Arranged so that the Field-Rivets May be Driven Readily.

One of the main reasons for the unsatisfactory condition of most of the older elevated railroads of this country is that their designers endeavored in every possible way to avoid field-riveting, in order to keep down the cost of erection; and in so doing they failed to develop the requisite amount of rigidity in the structures. The result is that many parts of those structures have to be either reinforced or removed, as can be seen at almost any time in New York City.

PRINCIPLE XXXI

Rivets Should Not be Used in Direct Tension.

In the days of iron rivets this was an important requirement, for the reason that the shanks were often so overstressed in cooling that the heads would fly off; but this does not occur with steel rivets. Nevertheless it is advisable to adhere to the rule, except for very unimportant members where there is a great excess in the number of rivets above the theoretical requirements.

PRINCIPLE XXXII

For Members of Any Importance Two Rivets do not Make an Adequate Connection.

For such details as lattice bars, of course, two rivets or even one rivet at each end will suffice; but where a direct calculable stress comes on the piece and only two rivets at each end are used, it will be found that they will work loose, while if three are employed, they will not, unless they are overstressed by the calculated stress on the piece.

PRINCIPLE XXXIII

Designs Must Invariably be Made so that All Metalwork after Erection Shall be Accessible to the Paint-Brush, Except, of course, Those Surfaces Which Are in Close Contact either with Each Other or with the Masonry.

This clause very properly cuts out the use of ordinary closed columns, which used to be a fruitful source of condemnation of old bridges.

PRINCIPLE XXXIV

In Multiple-Track Structures, if Any Bracing-Frames be Used between Panel Points or Bearings to Connect the Adjacent Stringers or Longitudinal Girders of Adjoining Tracks, they Must be Designed without Diagonals, in order to Prevent the Transference of Any Appreciable Portion of the Live Load from One Pair of Girders to Any Other Pair.

Such a transference would be doubly injurious, because it would throw on some of the girders more live load than they were proportioned to carry, and at the same time it would probably overstress the diagonals and their connections, and would certainly tend to distort laterally the flange angles of the longitudinal girders on account of the induced torsion.

PRINCIPLE XXXV

In Bridges, Trestles, and Elevated Railroads the Thrust from Braked Trains and the Traction Should be Carried from the Stringers or Longitudinal Girders to the Posts or Columns without Producing Any Horizontal Bending Moment of the Cross-Girders.

This is a requirement of the author's that has been employed in his designs for many years. Its correctness was established in his paper on Elevated Railroads.*

PRINCIPLE XXXVI

In Trestles and Elevated Railroads the Columns Should be Carried up to the Tops of the Cross-Girders or Longitudinal Girders and be Effectively Riveted Thereto.

The correctness of this proposition also was established in the said paper on Elevated Railroads.

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* This paper was originally published in the *Transactions* of the American Society of Civil Engineers in 1897, and was afterward incorporated in Mr. Harrington's compilation of the author's "Principal Professional Papers."

PRINCIPLE XXXVII

Every Column That Acts as a Beam also Should Have Solid Webs at Right Angles to Each Other, as No Reliance Can be Placed on Lacing to Carry a Transverse Load Down the Column.

The truth of this proposition is evident when one reflects that a single loose rivet or single bent lacing-bar in the whole line of lacing will prevent the latter from carrying as a web a transverse load. Loose rivets and bent lacing-bars are, unfortunately, not uncommon in structural metalwork.

PRINCIPLE XXXVIII

In Trestles and Elevated Railroads Every Column should be Anchored so Firmly to its Pedestal that Failure by Overturning or Rupture Could Not Occur in the Neighborhood of the Foot, if the Bent Were Tested to Destruction.

As long ago as 1891 the author designed pedestals which involved truly fixed ends for column feet; but it was several years later before such a detail began to come into general use. The ordinary old-style connection of columns to pedestals by an anchor-bolt at each of the four corners of the bed-plate is extremely weak and ineffective.

PRINCIPLE XXXIX

All Pedestals for Trestles, Viaducts, and Elevated Railroads should be Raised to such an Elevation as to Prevent the Accumulation of Dirt and Moisture about the Column Feet; and All Boxed Spaces in the Latter Should be Filled with Extra-Rich Portland-Cement Concrete.

The neglect of these precautions causes the rapid deterioration of the metal at bases of columns, and thus shortens the life of the structure.

PRINCIPLE XL

In Designing Short Members of Open-Webbed, Riveted Work, it is Better to Increase the Sectional Area of the Piece from Ten to Twenty-Five Per Cent than to Try to Develop the Theoretical Strength by Using Supplementary Angles at the Ends to Connect to the Plates.

This principle is based upon the results of some tests of the author's on the strength of single angles and pairs of angles attached by one leg only, by which it was found that 6″ × 3½″ angles connected by the long leg developed ninety per cent of the ultimate strength of a flat bar of equal net section, and that 3″ × 3″ angles connected by one leg only developed seventy-five per cent thereof.

PRINCIPLE XLI

Star-Struts formed of Two Angles with Occasional Short Pieces of Angle or Plate for Staying them Do Not Make Satisfactory Compression Members. Better Results are Obtained by Placing the Angles in the Form of a "T."

The truth of this statement was established by another series of experiments of the author's made at the same time as were the last-mentioned tests. The specimen columns did not develop on the average more than seventy-five per cent of the resistance they should have realized according to the usual straight-line formula for metal of the same tensile strength.

PRINCIPLE XLII

In Making Estimates of Weights of Metal the Computer Should Always be Liberal in Allowing for the Weight of Details

It is the author's experience that, in nearly every case, the weight of the finished structure exceeds slightly the estimated weight, and mainly on account of the use of more metal for details than was figured upon. Of course, if one sets out deliberately to "skin" a bridge so as to save all the metal he can, the actual weights of details may be made to underrun the estimate; but such a practice is most reprehensible.

PRINCIPLE XLIII

In the Design of Any Monolithic Concrete Structure, Proper Consideration Should be Given to the Stresses Resulting from the Continuity Thereof, and Reinforcement Should be Placed Wherever Required to Prevent the Formation of Unsightly or Dangerous Cracks.

It is not to be inferred from the above principle that the author is in favor of spending a great amount of time in an attempt to make an exact analysis of the stresses in the various parts of such a structure; but the approximate values of such stresses should be computed, and due provision should be made therefor.

PRINCIPLE XLIV

In Designing Any Reinforced Concrete Structure, Proper Consideration Should be Given to All the Construction Joints which will be Required; and if it is Found Impracticable to Avoid Shearing Stresses on any Such Joint, these Stresses Must be Fully Provided For, either by Keying or by the Use of Properly Placed Reinforcing Bars.

Reinforced concrete structures are usually monoliths; but construction joints are nearly always required at various stages of the work, and the concrete at such joints is to be relied on for little else than direct compression.

PRINCIPLE XLV

Care Must Be Taken to See that the Full Strength of Every Reinforcing Bar is Properly Developed at the Point where the Said Strength is Required.

PRINCIPLE XLVI

Sharp Bends in Reinforcing Bars Carrying Stress should Always be Avoided.

PRINCIPLE XLVII

Loose Stirrups Placed Diagonally in Reinforced Concrete Beams are of Very Little Value.

PRINCIPLE XLVIII

In General, Details Must Always be Proportioned to Resist Every Direct and Indirect Stress that May Ever Come upon them under Any Possible Condition without Subjecting Any Portion of their Material to a Stress Greater than the Legitimate Corresponding Working Stress.

This principle involves the whole theory of bridge detailing.

PRINCIPLE XLIX

The Science of Bridge Designing Lies Mainly in the Detailing.

If the reader who has perused carefully the preceding "principles" is not convinced of the correctness of this statement, he certainly will become so after a few years spent in making bridge computations.

PRINCIPLE L

There is But One Correct Method of Checking thoroughly the Entire Detailing of a Finished Design for a Structure, viz.: "Follow Each Stress Given on the Stress-Diagram from its Point of Application on One Main Member until it is Transferred Completely either to Other Main Members or to the Substructure, and See That Each Plate, Pin, Rivet, or Other Detail by which it Travels has Sufficient Strength in Every Particular to Resist Properly the Stress that it thus Carries; Check also the Sizes of such Parts as Stay-Plates and Lacing, which Are Not Affected by the Stresses Given on the Diagram, and See that the Said Sizes are in Conformity with the Best Modern Practice."

But to do all this as it should be done necessitates the computer's being, in the truest sense of the term, an expert of the highest order in all branches of bridge designing.

CHAPTER XVI

DETAILING IN GENERAL

As stated near the end of the preceding chapter, the science of bridge designing lies mainly in the detailing. This is an axiom the correctness of which is fully recognized by the bridge expert, but, unfortunately, not by many railroad men and other purchasers of steel structures; for if it were, they would never entrust the designing of the bridges they buy to the manufacturers, whose usual custom it is to have stress-sheets prepared by their comparatively high-priced computers and to turn over the detailing to cheap draftsmen. Is it not folly to spend a lot of money in making a bridge heavy in the main members and at the same time to pay so little attention to detailing that, were the structure tested to destruction, it would fail in the connections long before developing the full strength of the main members? Yet such is commonly the custom among the purchasers of bridges, who think that there is no need to retain the services of a bridge specialist to prepare the plans when they can get such work done for nothing (!) by the bridge-manufacturing companies. They do not recognize that the labor of preparing the plans has to be paid for by some one, and that when it is done by the contractor the cost is hidden in the pound price of the completed metal-work.

It is, unfortunately, not only the purchasers of bridges who are ignorant of the important axiom with which this chapter is introduced; for even such scientific and practical men as the editors of technical papers are occasionally just as badly informed. A good illustration of this was given some years ago in *Indian Engineering,* when the editor of that well-known paper wrote as follows: "Now, however, the standardizing of parts—the provision by manufacturers of pieces calculated for a given load—and such like has so far simplified matters that any one can put together the details of a simple bridge design." Unfortunately for the editor, there appeared in the same issue a detailed illustration and description of the Koli Bridge, which furnished a most forcible contradiction of the statement. The author seized the opportunity to call attention to the editor's error by furnishing an analysis of some of the secondary and induced stresses in the structure illustrated, pointing out the following important defects:

First. The span of one hundred and twenty-eight (128) feet consisted of two *pony trusses* with six (6) side braces per side and six (6) light overhead arch-struts that riveted only to the upper surfaces of the top chords.

Second. The truss depth employed was just about one-half that required for greatest economy of metal, as was shown by computing the weight of the chords and that of the web, which two weights should be about equal for minimum total weight of metal in trusses.

Third. There was really no upper lateral system at all, for struts without diagonals cannot form a lateral system.

Fourth. Only two out of the ten panels of each truss were counterbraced.

Fifth. All web diagonals were composed of flat bars, and the counters had no adjustment. (It must be remembered that the trusses were of the riveted type.)

Sixth. The detailing of the vertical posts was absurd, for there were no stay-plates at the ends of the lacing system, and the bars of the latter had a far-too-small inclination to the vertical.

Seventh. The chords were strengthened toward the centre of the span by piling on an extra cover plate instead of by thickening the webs of the channels.

Eighth. The bottom chords were closed troughs that would hold water and ensure rusting the metal rapidly.

Ninth. The bottom chords were loaded transversely at mid-panel, thus ensuring the greatest possible bending moments upon them.

Tenth. The floor-beams were attached to the inner channels of the bottom chords, and there were no means provided for transferring a portion of the reaction to the outer channels, thus making the inner ones do practically all the work of resisting bending.

Eleventh. At the fixed end of the span there was no hinged pedestal, consequently the distribution of load over the shoe was far from uniform.

Twelfth. The eccentricity in the connections of web members to chords was both excessive and uncalled-for, thus making the secondary stresses exceedingly high.

It may seem at first thought that it was hardly necessary to go abroad for an example of the evil effects of leaving the detailing to incompetent designers and to parties interested in making money out of the construction, but not out of the use of the structure. The reason is that the example was at hand and is a glaring one. It represents, though, in a way, the errors in American practice of a decade earlier than the date of building the bridge referred to. To-day the detailing of bridges in general is improved materially, this result having been accomplished primarily by the independent designers, whose work has been copied more or less by the manufacturing companies.

There is no need to treat here of the faulty bridge detailing of the distant past, because that, in the main, has been corrected in modern designs. The reader who is interested in the matter is referred to the author's paper on "Some Disputed Points in Railway Bridge Designing," published in the *Transactions* of the American Society of Civil Engineers for February and March, 1892, and reproduced in Mr. Harrington's "Principal Professional Papers."

In railway bridge designing, for a number of years the average ratio of weight of details to weight of main members has been gradually increasing; and the end is not yet, because the average bridge designer has still a great deal to learn concerning the importance of good and efficient detailing. As long as contracts for bridges are awarded to bridge companies on competitive designs, and the structures are paid for by the lump sum instead of by the pound, just so long will the science of detailing be ignored, and just so long will bridges be built which will eventually wear out, simply for want of a little more metal distributed just where it is needed, viz., in the details.

The author feels that he cannot speak too forcibly concerning the importance of thoroughly scientific detailing for all kinds of metal work; for what avails it that a structure have an excess of section in every main member if a single important detail be lacking in strength? If the author were in a position where he had to cut down the weight of a structure even as much as thirty per cent he would unhesitatingly take the metal almost entirely out of the sections of the main members and leave the detailing practically unchanged. A structure thus designed would long outlast one of the same type in which the weight of the details and that of the main members were reduced in the same proportion.

The designer who is desirous of improving his detailing should begin by studying so thoroughly the "First Principles of Designing," given in the preceding chapter, that he will have them permanently in mind; then he should read Chapters XIX to XXIV, inclusive, and learn how the principles are applied. In those chapters are described practically all the important bridge details in common use, while the remainder of this chapter will be devoted to the consideration of a number of features of general application which the designer should constantly bear in mind.

Attention is called to the detail illustrated in Fig. *22fff* for carrying either the thrust of a braked train, or the horizontal reaction of all the driving wheels, from stringers to trusses without bending the cross-girders. This detail is only now coming into general use, although the author has been employing it in his designs for more than two decades. Many designers think that the thrust of a braked train comes so seldom on any structure and its effect on the cross-girders is so small that it may be ignored; but this idea is incorrect, because bridges in some locations are subjected to train thrust constantly, and every bridge is liable to receive it at any time. Moreover, if one will figure the effect on the flanges of the cross-girders, which have transversely a very small moment of inertia, he will be surprised to find how high the extreme fibre stress will run. For instance, taking a case that is far from being extreme and which, consequently, is unfavorable to the illustration, let us assume an average live load of 7,500 pounds per lineal foot for a thirty (30) foot panel, and that the coefficient of friction is 0.2, making the thrust on the cross-girder 45,000 pounds. If the length of the girder is seventeen (17) feet, and if there are four lines of stringers, the transverse bending moment will be $22{,}500\left(\frac{17-5}{2}\right) = 135{,}000$ ft.-lbs. $= 1{,}620{,}000$ in.-lbs. Assume this to be divided equally between the two flanges, giving 810,000 in.-lbs. per flange, and that each flange is composed of two $6'' \times 6'' \times 9/16''$ angles, the web thickness being $\frac{3}{8}''$. The moment of inertia of the two angles is about 90, consequently the extreme fibre stress will be $\frac{810{,}000 \times 6.2}{90} =$ 55,800 lbs. nearly. Of course, the partial continuity of the rails and guard rails will often tend to transfer some of the train thrust directly to the abutments and to the embankment, but when the approach is a wooden trestle, it will have its own share of thrust to take care of, consequently too much reliance should not be placed on this assumed relief. It is far better to detail the lateral system so that the thrust is not carried at all by the cross-girders, especially as the extra cost of so doing is trifling.

The proper detailing of the lateral system so that the train thrust is taken directly to the trusses or girders has the added advantage of relieving the floor-beam flanges from the bending which will be occasioned by the changes in lengths of the chords under load. As was shown in Chapter XI, the stresses in the flanges of the floor-beams from this cause are very high, unless provision is made, as above outlined, to relieve them.

Web members composed of four angles and carrying mainly tension stresses may very properly be stayed by occasional stay plates instead of by a system of lacing. This is a legitimate economy, but it should not be carried so far as to apply to members that are mainly struts. Hangers carrying floor-beams riveted to one side, however, should be laced.

In riveted bridges due attention is not always given to the strength of connecting plates, which are sometimes made so thin that they would rupture along the lines of the rivet holes before developing the full strength of the attached web member. This feature of detailing is exemplified in Chapter XXII.

A very faulty detail in riveted spans, far too common in current practice, is the placing of the pedestal pins below the bottom chords instead of on their centre lines. The error involved is the ignoring of the bending effect on the inclined end posts and the end panels of bottom chords due to the moment of the train thrust. Of course, this could be provided for by strengthening these main members, but to do so would be uneconomic, because it is easier and better to place the pin on the centre line of chord where it belongs.

A faulty detail that is still in existence, although it should long ago have been relegated into oblivion, is the failure to carry pin plates of columns well beyond the edges of the batten plates. Neglecting to do so materially weakens the column, as has been shown by full-size column tests. The lap for this detail should be great enough to provide for two or three rows of rivets. Similarly, in riveted structures the batten or stay plates should extend well between the connecting plates which attach the main member under consideration to the other main members.

Forked ends of columns are often, without any good reason, made weaker than the member of which they form a part. Whenever it is practicable, there should be placed between the projecting pin plates a diaphragm composed of a plate and angles, and it should be carried as close to the pin as the connection of other members will permit. If it be impracticable to use the diaphragm, the pin plates should be so reinforced that their ratio of length to least radius of gyration will not exceed the corresponding ratio for the strut itself, and so that the allowable intensity for forked-ends will not be exceeded.

Batten plates are often made too short. Their length should never be less than their width, except in bracing members, and often it should be greater. Wide batten plates with only two or three rivets on a side are an absurdity, and their use is an indication of either ignorance or dishonesty.

The lacing of struts is often altogether too light. For members of ordinary section the rules given in Chapter LXXVIII will apply; and for unusually large sections it is better to dispense with lacing entirely and adopt a cover plate above and one below. This forms the strut into a closed column, hence provision must be made for the admission of a man for painting the interior. Once in designing a large bridge the author employed box compression chords, of the form shown in Fig. 22*ddd;* but the innovation thus involved was so great that the manufacturers managed to succeed in having the section changed to the one ordinarily employed. This detail has since been adopted on the Hell Gate Arch Bridge.

The rational design of lacing is really impossible. The formula used in the specifications of Chapter LXXVIII is strictly empirical in form, but is probably as good as any. It makes the strength of the lacing depend upon the load on the column and upon the ratio of its length to radius of gyration in the direction of the lacing, which two factors are undoubtedly the most important ones involved in the matter. The limiting thicknesses specified are those which experience has shown to be the proper values.

In general, it will be best to employ single-bar lacing at 60-degree inclination for small members, and single-angle lacing at the same inclination for larger ones. For such lacing the stress in each bar or angle will be 1.16 times the shear carried by the lacing; and the stress in the connecting rivets will have this same value, no matter whether adjacent bars are connected by the same rivets or by different ones. For double lacing at 45-degree inclination, the stress in each bar is seven-tenths (0.7) of the shear carried by the lacing, and the stress in the connecting rivets will have the same value when adjacent bars are not connected by the same rivets; but when adjacent bars are attached by the same rivet or rivets, the stress in the rivets will be equal to the shear carried by the lacing. In the latter case, the shearing strength of the rivets will govern when all of the bars are either inside or outside of the member; but their bearing on the portion of the main section through which they pass will govern when the bars are placed alternately within and without the member.

A bar 2½″ × 5/16″ in section has a strength equal to the single-shear value of a ⅞″ shop rivet—6,000 pounds—when the ratio of its length to its thickness is forty (40). From this fact the strength of any lacing bar can be readily determined. In figuring the compressive strength of bars in a double-lacing system, when the intersecting bars are riveted together at the crossing, the ratio of length to thickness should be taken as only two-thirds of the actual value, to allow for the strengthening effect of the central connection. This is in accordance with the standard practice of allowing the ratio of length to thickness to be fifty (50) per cent greater for bars of double lacing than for bars of single lacing. The strength of angles used in lacing will rarely, if ever, need testing.

It follows from the above that single lacing at 60-degree inclination, with bars ⅜″ or more in thickness fastened by one ⅞″ rivet at each end, will carry a shear of 6,000 ÷ 1.16, or 5,200 pounds; while double lacing at 45-degree inclination, with bars 5/16″ or more in thickness likewise fastened with one ⅞″ rivet, will carry a shear of 6,000 pounds. For shears up to 5,200 pounds, single lacing of 2½″ bars should, therefore, be employed; and for shears between 5,200 pounds and 6,000 pounds, double lacing of 2½″ bars. For shears exceeding the latter figure, lacing of 5″ bars or some form of angle lacing should be adopted.

Angle lacing is usually lighter than bar lacing for wide members, particularly if no gusset plates for the lacing are used. It is also preferable because its stiffness helps to prevent the tendency of members to warp into a diamond shape under fabrication. From this view-point it is better to attach the angles to the flanges directly without the use of gusset plates.

It is always advisable, when possible, to make adjacent lacing bars or angles intersect on the gauge lines of the flange angles, in order to avoid the bending which would be caused by an eccentric connection. Bar lacing should always thus intersect, and it is preferable that angle lacing do so as well. This end can be attained for angle lacing by attaching the angles to gusset plates, or, if the flange angles of the member are turned inward, by putting alternate angles on the inside and on the outside. The appearance of the latter arrangement is not particularly good, however. All things considered, the author considers it best to connect angle lacing to gusset plates.

Bar lacing is usually put on the outside of a member, but angle lacing should preferably be put on the inside, as otherwise the outstanding legs will interfere with handling; and they are very likely to be bent.

Fig. 16*a* can be employed to find the weights of bar lacing—the upper portion for double lacing at 45-degree inclination, and the lower portion for single lacing at 60-degree inclination—for various distances from centre to centre of rivet lines. The weights are given for bars 2½" wide; and those for bars 5" wide are just twice as great. The full portions of the curves correspond to thicknesses which can be used for main members, and the dotted portions to thicknesses allowable for bracing members only. The weights of double-bar lacing (or latticing) at 60-degree inclination would be double those for single-bar lacing at the same inclination; and the weights of single-bar lacing at 45-degree inclination would be half of those for double-bar lacing at the latter inclination. The curves assume that the bars are lapped at intersections, which detail should always be adopted for bar lacing.

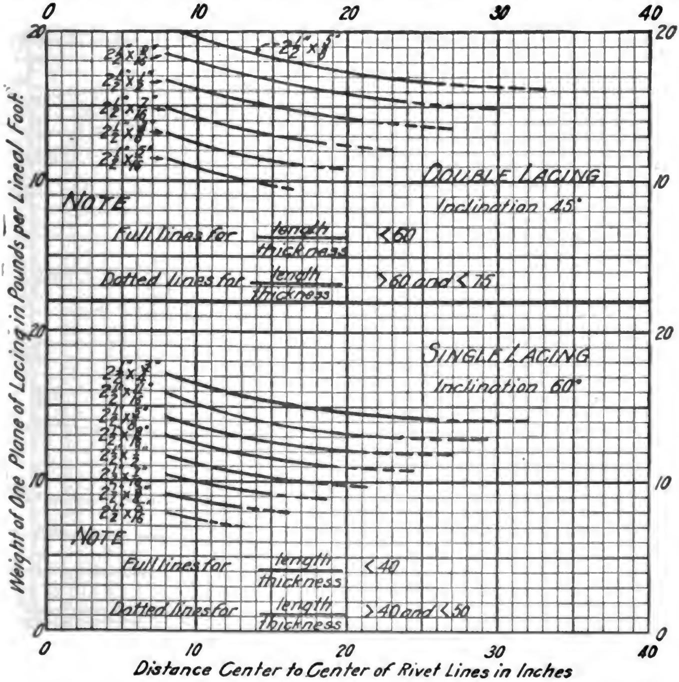


Fig. 16*a*. Weights of Bar Lacing.

Fig. 16*b* gives weights of single-angle lacing at 60-degree inclination.

The upper portion is to be used when gusset plates are employed, and

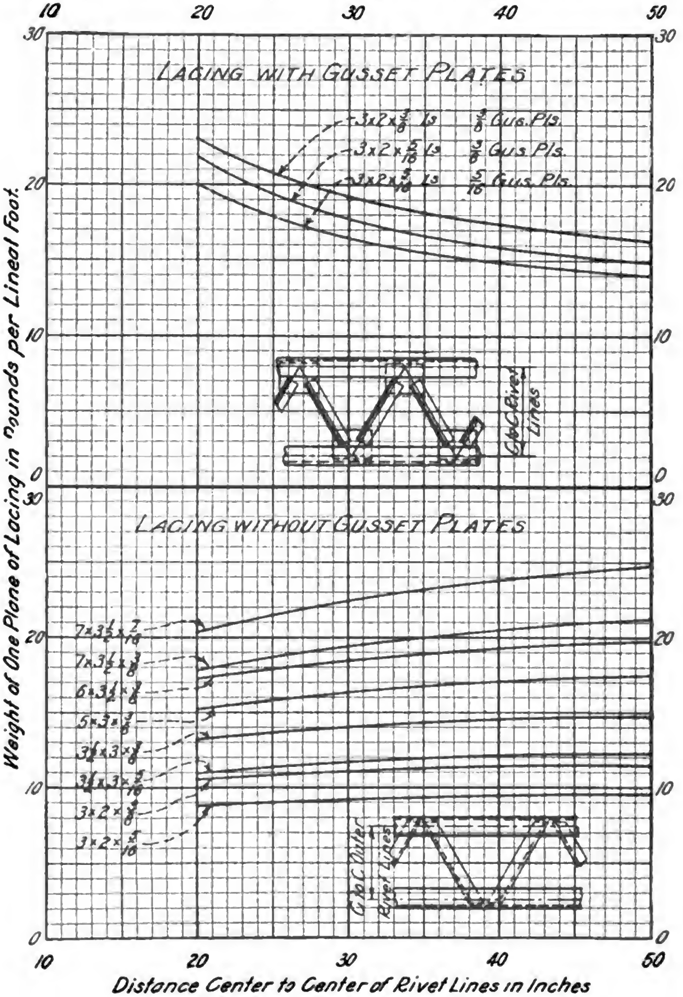


Fig. 16b. Weights of Angle Lacing.

the lower portion when there are no gusset plates and when the angles are arranged as shown in the sketch. It will be noted that the curves are to be entered with the distance from centre to centre of outside rivet gauges of the main member in all cases.

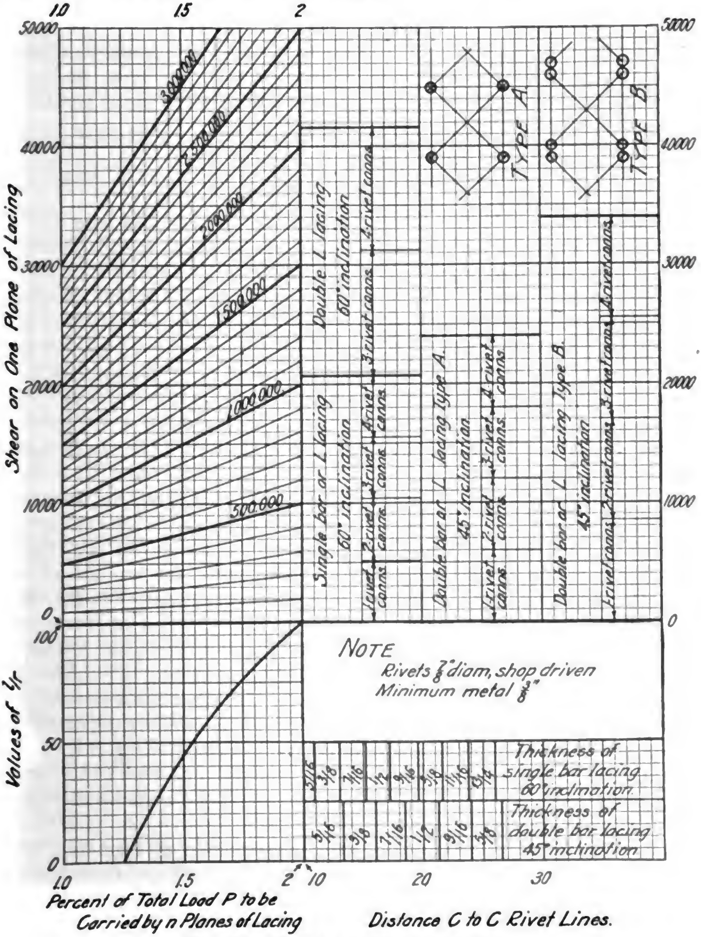


Fig. 16c. Diagram for Designing Lacing.

NOTE.—The diagonal lines in the upper left hand portion of the diagram represent values of $\frac{P}{n}$.

Fig. 16c can be utilized for the design of lacing. The lower left hand portion is a graphical solution of the equation,

$$S = \frac{200P}{16{,}000 - 60\,\frac{l}{r}},$$

which appears in Clause 71 of Chapter LXXVIII. The curve gives values of the shear S to be carried by the lacing in percentages of P, the total load on the member; and by tracing vertically into the upper left-hand portion there can be found the corresponding value of the shear to be carried by one plane of lacing for any value of $\frac{P}{n}$, where n equals the number of planes of lacing. On then projecting horizontally over to the upper right-hand portion of the diagram, there can be found the number of ⅞″ shop rivets required in the connections of the following types of lacing: single-bar or single-angle lacing at 60-degree inclination; double-angle lacing at 60-degree inclination; Type A double-bar or double-angle lacing at 45-degree inclination, in which the connecting rivets pass through both bars at intersections; and Type B double-bar or double-angle lacing, in which adjacent bars are not connected by the same rivets. For Type A lacing it is assumed that all bars are on either the inside or the outside of the member; and if they are alternately within and without, the strength of any rivet connection will vary from ten per cent greater than that given for Type A when the rivets pass through ⅜″ metal in the main member, to that given for Type B when the thickness of the said metal is ½″ or more. The diagram assumes that all metal is at least ⅜″ thick. For 5/16″ metal in either lacing bars or main member the strength of 60-degree lacing and that of Type B 45-degree lacing will be ten per cent less than the values indicated. For Type A 45-degree lacing, there will be the same reduction in strength when the metal of the main section is 5/16″ thick, but none when the lacing bars or angles are 5/16″ thick and the main member ⅜″ or more in thickness. When bar lacing is used, the thickness required can be found by means of the lower right portion of the diagram. The relative economy of bar and angle lacing can be determined by means of Figs. 16*a* and 16*b*. Bar lacing is more easily fabricated than angle lacing, and will be cheaper unless the angle lacing effects a considerable reduction in weight.

The following examples illustrate the application of Figs. 16*a*, 16*b*, and 16*c*.

First. A compression member composed of two built channels with angles turned inward and laced on two sides carries a load of 500,000 pounds, the value of $\frac{l}{r}$ about an axis perpendicular to the planes of the lacing being 75, and the distance between the lines of rivets by which the lacing is fastened being 15″. What lacing should be used when ⅞″ diameter rivets are employed?

In this case $P = 500{,}000$ and $n = 2$, so that $\frac{P}{n} = 250{,}000$. Entering at the lower left-hand margin of Fig. 16*c* with $\frac{l}{r} = 75$, we pass horizontally to the heavy diagonal curve, and thence vertically to a point midway between the diagonal lines for $\frac{P}{n} = 200{,}000$ and $\frac{P}{n} = 300{,}000$. We then read the value of the shear on one plane of lacing as 4,300 pounds; and projecting horizontally to the right, we find that one-rivet connections will suffice for either single or double lacing. Entering now the lower right-hand portion of the diagram with 15″ as the distance between rivet lines, we find the required thickness of single-bar lacing at 60-degree inclination to be 7/16″, and of double-bar lacing at 45-degree inclination to be 3/8″. By Fig. 16*a*, the weight of single lacing of 2½″ × 7/16″ bars is 9.2 pounds per lineal foot, and that of double lacing of 2½″ × 3/8″ bars is 11.4 pounds. Evidently the single lacing will be the better.

Second. Suppose that in the first problem the load on the member had been 650,000 pounds, the other factors remaining unchanged. What lacing should be used?

In this case $\frac{P}{n} = 325{,}000$. Entering as before with $\frac{}{r} = 75$, we trace horizontally to the heavy diagonal curve, and then vertically to a value of $\frac{P}{n} = 325{,}000$, taken by locating a point about one-quarter of the distance between the lines for $\frac{P}{n} = 300{,}000$ and for $\frac{P}{n} = 400{,}000$. Then on tracing horizontally over into the right-hand portion of the figure, we find that one-rivet connections will suffice for double lacing at 45-degree inclination, but that two-rivet connections will be needed for single lacing at 60-degree inclination. If we adopt bar lacing, we shall, therefore, have to use 5″ × 7/16″ single lacing, weighing 18.4 pounds per lineal foot, or 2½″ × 3/8″ double lacing weighing 11.4 pounds per lineal foot. The latter is evidently the better of the two.

Third. Suppose that in the second problem the distance from centre to centre of rivet lines had been 25″, the other factors remaining unchanged. What type of lacing would be the best?

In this case single-bar lacing at 60-degree inclination would require the use of 5″ × ¾″ bars weighing 28.5 pounds per lineal foot, and double-bar lacing at 45-degree inclination would necessitate 2½″ × 5/8″ bars weighing 16.7 pounds per lineal foot. Turning now to Fig. 16*b*, we find that single-angle lacing with gussets will weigh 20.8 pounds per lineal foot, if 3/8″ metal be used, or 17.9 pounds, if 5/16″ metal be employed. If the angles of the main members should be 6″ wide, single lacing of 3″ × 2″

angles without gusset plates at 60-degree inclination could be adopted, because two-rivet connections could be obtained. The weight will be about 10.9 pounds per lineal foot, if $\frac{3}{8}''$ metal be used, or 9 pounds, if $\frac{5}{16}''$ metal be employed. This latter type of lacing would evidently be the lightest of any.

Fourth. A top chord member composed of a cover-plate and two built channels, with lacing on the bottom side, carries a load of 2,200,000 pounds. What lacing should be employed, the $\frac{l}{r}$ of the member about a vertical axis being 40, and the distance between rivet lines by which the lacing is attached being 30″?

In this case n equals 2, since the cover-plate is to be considered as one plane of lacing. The value of $\frac{P}{n}$ is, therefore, 1,100,000 pounds. Entering with $\frac{l}{r} = 40$, tracing over horizontally to the heavy diagonal curve, thence vertically upward to the line for $\frac{P}{n} = 1{,}100{,}000$, and then horizontally to the right, we find that we can use the following types of lacing: single-bar or single-angle lacing at 60-degree inclination with 4-rivet connections; double-bar or double-angle lacing at 60-degree inclination with 2-rivet connections; double-bar or double-angle lacing, Type *A*, at 45-degree inclination with 3-rivet connections; or double-bar or double-angle lacing, Type *B*, at 45-degree inclination with 2-rivet connections. Evidently angle lacing should be employed, on account of the great distance between rivet lines. For the single-angle lacing we can adopt 6″ × 3½″ × $\frac{3}{8}''$ ∟'s without gusset plates; and entering Fig. 16*b* with the distance from centre to centre of rivet gauges as 30″, we find that this type will weigh 18.4 pounds per lineal foot. For the double-angle lacing at 60-degree inclination we can use 3″ × 2″ × $\frac{3}{8}''$ ∟'s, which with $\frac{3}{8}''$ gusset plates will weigh 2 × 19.2 = 38.4 pounds per lineal foot; and without gusset plates, 2 × 11 = 22 pounds per lineal foot. Type *A* double lacing at 45-degree inclination would hardly ever be used in such a case, on account of the larger angle required and the difficulty of placing the rivets in the connection. For Type *B* double lacing at 45-degree inclination we could use 3″ × 2″ × $\frac{3}{8}''$ ∟'s, either with or without gusset plates; and they will evidently weigh somewhat less than the same form of lacing at 60-degree inclination. The single lacing of 6″ × 3½″ × $\frac{3}{8}''$ ∟'s would certainly be the best type to employ.

Splices in compression chords are often very faulty, in that too much reliance is placed on the perfection of shop-work in abutting ends. While it is practicable to obtain pretty fair contact for such ends in struts of ordinary dimensions, the author believes that in those of unusual size it is not, hence the deeper or wider the piece the greater should be the proportionate strength of the splice. For this reason he favors making the minimum strength of such splices six-tenths (0.6) of the strength of the member spliced; and in large struts he advises a still greater limit. In fact, in pin-connected bridges of great span length he would advocate making all chord members truly hinged at the panel points; and in large riveted bridges he places the joints there so as to get the benefit of the great strength and stiffness of the connecting plates, thus economizing on metal.

It is a mistake to omit end floor-beams of bridges. This has been forcibly stated in print before, and has been endorsed by the profession, nevertheless the faulty practice is still very common. When they are omitted and replaced by horizontal struts, the latter are often so eccentrically connected that they are stressed beyond all reasonable limits, or would be if they received their computed direct stresses.

The connection of transverse struts to the upper surface only of top chords is a most reprehensible practice. Granting that the secondary stresses due to eccentric connection are not large enough to be serious (which is true if the calculated greatest stress on the transverse strut be very small), it is still necessary, in order to make the chords fixed in position at the panel points, that the lateral struts there should take hold of the said chords symmetrically above and below and should stiffen them effectively by large connecting plates.

In designing members composed of one or two angles, care must be taken to reduce their effective values by the percentages given in the specifications of Chapter LXXVIII. These percentages are intended to provide for the fact that the stresses are usually applied to such members more or less eccentrically, and they agree in general with the values found in certain tests which have been made. When a member consists of two angles riveted back to back, with a plate riveted to their outstanding legs, the efficiency of the angles should be taken as for the two-angle section, while that of the plate should be considered unity.

Knee-braces, extending from vertical posts to upper lateral struts, are often attached to both members at points which are not stayed by batten plates, the effect being to cripple the lacing before the working strength of the knee is developed. If the knee takes hold of the post on the central plane there should be a diaphragm connecting opposite webs of the post; but if it grips the transverse faces, there should be a properly disposed batten plate on each face.

In order to avoid the use of very large connecting plates for lateral diagonals, the latter are often so laid out that their axes intersect at some distance from the central plane of truss, thus inducing large bending moments by eccentricity, which is an exceedingly bad feature, unless the said bending moments be properly provided for.

Even nowadays portal bracing is sometimes designed more for appearance than for strength and rigidity. Portals involving diagonal adjustable rods connected to a central ring are an absurdity, yet a well-known bridge engineer of long experience was guilty of making them—and no longer ago than 1897 he employed this abortive detailing in the design for a 400-foot, double-track-railway span.

Stringer bracing diagonals are sometimes made of such small sections that their values of $\frac{l}{r}$ must have been ignored altogether, or else the ends were considered fixed, a condition which merely connecting to an unstiffened side plate cannot possibly produce.

In computing the bending moment upon the inclined end post of a through span, it is necessary to know the position of the point of contraflexure. This can be obtained from Fig. 16*d* for two conditions of fixedness and for various values of the ratio of the length of the upper braced portion of the member to that of the lower unbraced portion. The formula for the upper curve is demonstrated in "Modern Framed Structures," and that of the lower one was evolved a few years ago in the author's office. Ordinarily, the value of *n* varies from 0.25 to 0.75, averaging about 0.5; for which $k = 0.555$ for the lower curve, and 0.57 for the upper one. In the old days of his practice, when he did most of his own computing, the author used to assume the value to be 0.5, although he knew

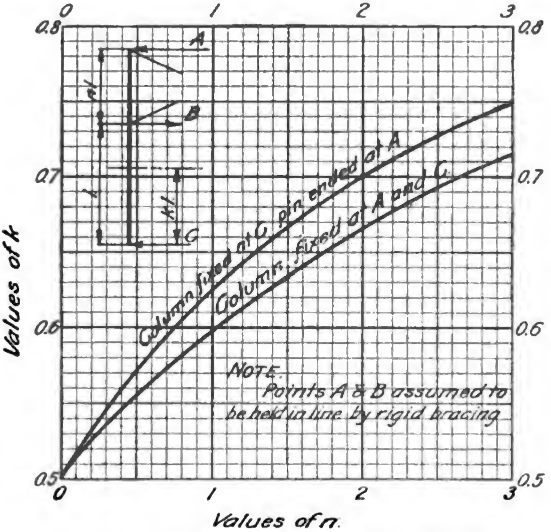


Fig. 16*d*. Points of Contraflexure in Braced Columns.

that the assumption was not strictly correct. The error involved was about ten per cent of the value of the bending effect, which error amounts generally to less than three per cent of the value of the combined bending and direct stress effect. In heavily loaded railroad bridges the permis-

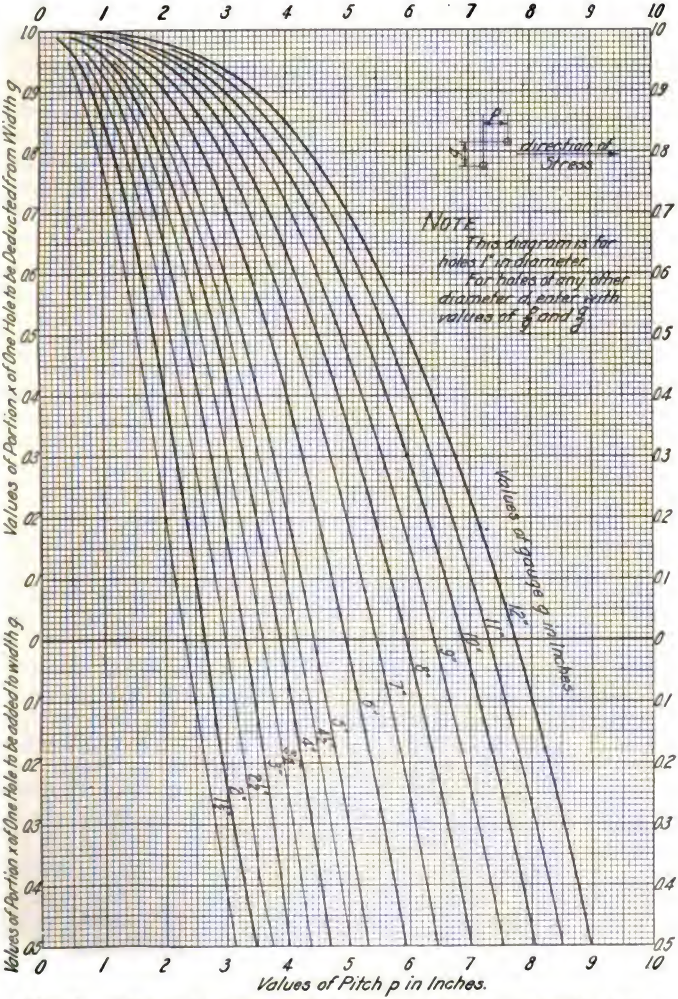


Fig. 16e. Diagram for Calculating Net Sections of Riveted Tension Members.

sible increase of thirty (30) per cent in the intensity due to wind loading causes the effect of the loading without wind to be the governing condition; but in lightly loaded railroad bridges the opposite is the case. In view of the fact that the point of contraflexure can now be ascertained at a glance and without any necessity for figuring, it is better to adopt for the bending-moment computation the more correct lever arm.

In designing riveted tension members care must be taken to see that all net sections are properly figured along diagonal as well as transverse lines. As the computation of the diagonal sections for combined shear and tension is very tedious, Fig. 16e is given to facilitate the work. The formula on which this diagram is based was worked up originally in the author's office by his chief computer, V. H. Cochrane, Esq., C.E., and was published by that gentleman in *Engineering News*, April 23, 1908; but the complete diagram was first presented by T. A. Smith, Esq., C.E., in *Engineering News* of May 6, 1915.

The curves of this diagram give the portion x of one rivet hole which is to be deducted from or added to the distance g between the gauge lines of the two rivets in order to determine the net effective length of the metal in this width. Referring to the diagram, we find that for $p = 0$, $x = 1$. This is evidently correct, since in that case we should subtract two half holes. As p increases, x decreases, reaching zero at some particular value (as for $g = 3''$ and $p = 3.3''$), at which point the effective length on the diagonal is just equal to the width g; and as p becomes still larger, the effective length on the diagonal becomes greater than the width g, so that x is additive. The lower portion of the diagram, in which x is additive, will be found of value for unusual combinations only.

If a plate has several lines of rivets therein, the total number of holes to be counted out of any section is equal to $1 + x_1 + x_2 + x_3 +$ &c. The term unity represents the deduction for the outer halves of the rivets on the two outside gauge lines, and the quantities x_1, x_2, etc., denote the deductions for the various intermediate gauge widths.

The diagram can be applied to any arrangement of rivets. Consider, for example, the group shown in Fig. 16*f*, supposing the diameter of the holes to be one inch. Evidently the plate might rupture along the lines 1–3, 2–4, 1–4, 2–3, 1–2–4, 1–3–4, 1–3–5, 2–3–4, 2–3–5, 1–2–3–4, or 1–2–3–5. Since several of these sections are identical, and others are evidently not the weakest, it will be sufficient to test the lines 1–3, 1–3–5, and 1–2–3–5. For the line 1–3, we evidently count out two holes. For the line 1–3–5, the deduction required is $1 + 1 + 0.43 = 2.43$ holes. For the line 1–2–3–5, we must count out $1 + 0.17 + 0.71 + 0.43 = 2.31$ holes. The net effective width of the plate is therefore $16'' - 2.43'' = 13.57''$. The minimum pitch at which we need to take out two holes only is 3.3″, for which pitch the deduction on the line 1–3–5 would be $1 + 1 + 0 = 2$ holes. If the pitches should be made 3½″, the deduction on the line 1–3–5 would be $1 + 1 - 0.12 = 1.88$, and that on the line 1–2–3, $1 - 0.12 + 0.58 = 1.46$.

If a plate has only two lines of rivets therein, we evidently need to count out but one rivet hole when p is large enough to make $x = 0$.

With rivets arranged as in Fig. 16*g*, and $g_1 = g_2$, no portion of Hole 2 will be counted out whenever each of the x's is 0.5 (or less); for in this case we must take two holes out of line 1–3, and $1 + 0.5 + 0.5 = 2$ holes out of line 1–2–3. When g_1 and g_2 are unequal, no part of Hole 2 will be counted out when $x_1 + x_2$ is equal to or less than 1.0. This value of p can be determined most easily from Fig. 16*e* by locating a vertical line which cuts the gauge lines g_1 and g_2 at equal distances above and below the line $x = 0.5$.

In figuring the net sections of an angle having rivets in both legs, the distance g between rivets in the opposite legs can be taken as the sum of the distances of the said rivets from the back of the angle.

The above discussion has considered the finding of the net section at any one point of a member only. However, it is usually necessary to consider adjacent sections as well. For instance, if there should be two rivet holes in a plate at one section, and six holes in the same plate at a section a foot away, the net section at the first point could not be taken greater than that with six holes out plus the amount that could be developed by the rivets in the intervening foot of distance. Also the strength at a section near the end of a member or any component part thereof must not be taken as greater than the amount which can be developed by the rivets between the section and the end of the said member or component.

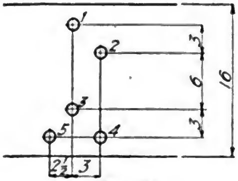


Fig. 16*f*.

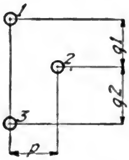


Fig. 16*g*.

Rivet Groups in Tension Members.

As an actual example of the possible error in the figuring of net sections, the following case, which has lately come to the author's notice, may be cited. One leaf of a tension member of a truss consisted of one 20″ × ½″ plate, one 12″ × ½″ plate, and two angles. The angles were fastened to the 20″ plate by rivets spaced 6″ centres, and there was a third rivet at the centre of the plate in line with those through the angles. The 12″ plate was also tacked to the 20″ plate by 2 rivets every 6″, these rivets being staggered with those first mentioned. It was proposed to figure the net section of the plates by deducting three holes from the 20″ plate and one from the 12″ plate, or by deducting two holes from each plate. However, since the stress could not possibly transfer back and forth from one plate to the other, evidently the correct method of figuring was to deduct three holes from the 20″ plate and two from the 12″ one.

A tie plate on a member is frequently figured to make up for extra holes in the member at the point where it occurs. While this device may be legitimate when the tie plate is fairly long, it is not justifiable when it is but 9″ or 12″ in length.

The riveting in the body of a member should be laid out in such a manner as to reduce the gross area as little as possible. At splices and connections, however, it will be necessary to have many more holes than in the body of the member. The riveting at the ends of the splice plates and the connecting plates should be carefully detailed, in order to make the splice as short as possible, and at the same time to avoid under-cutting the required strength of the member at any section. To do this it will generally be necessary to compute the strength at each line of rivets near the end of the connection.

It frequently happens that a group of rivets is subjected to a moment, which may or may not be accompanied by a direct load. It then becomes necessary to determine the stresses in the various rivets in the group.

Suppose the rivet group to be acted upon by a load P in the same plane therewith, having an eccentricity e with respect to the centre of gravity of the group; or, what amounts to the same thing, by a central load P and a moment M. Then the average direct stress S_d in each rivet will be

$$S_d = \frac{P}{N}, \qquad \text{[Eq. 1]}$$

where N is the number of rivets in the group. The moment M will produce in any rivet a stress S_b, the value of which depends on its distance from the centre of rotation and is given by the formula,

$$S_b = \frac{Mr}{I}, \qquad \text{[Eq. 2]}$$

in which r is the distance of the rivet from the centre of gravity of the group, and I the polar moment of inertia of the group about the said centre of gravity on the assumption that the area of each rivet is unity. The value of I is given by the expression,

$$I = \Sigma r^2. \qquad \text{[Eq. 3]}$$

If now we take the centre of gravity of the group as the origin of coordinates, the value of I will be

$$I = \Sigma x^2 + \Sigma y^2; \qquad \text{[Eq. 4]}$$

and the components S_y and S_x of S_b will be given by the formulæ,

$$S_y = \frac{Mx}{I} = \frac{Mx}{\Sigma x^2 + \Sigma y^2}, \qquad \text{[Eq. 5]}$$

and

$$S_x = \frac{My}{I} = \frac{My}{\Sigma x^2 + \Sigma y^2}. \qquad \text{[Eq. 6]}$$

In nearly all cases the rivets will be arranged in rows parallel to or perpendicular to the direction of the line of action of P, and will be spaced nearly, or quite, evenly throughout each row. Suppose the group to consist of m rows parallel to the line of action of P, with n rivets in each row. Also, let the spacing of the various rows be constant and equal to s, and the pitch of the rivets in each row be uniform and equal to p. Further, assume the x axis to be perpendicular to the line of action of P. The moment of inertia I of the group about the centre of gravity can be found by getting the moment of inertia of the rectangle having the sides ms and np, subtracting from this quantity the moments of inertia of each of the small rectangles of area ps about its own centre, and then dividing the result by ps, whence we obtain the formula,

$$I = \frac{nm}{12}\left[(n^2 - 1)p^2 + (m^2 - 1)s^2\right]. \qquad \text{[Eq. 7]}$$

For large groups it will be sufficiently accurate to use for the value of I the expression,

$$I = \frac{nm}{12}(n^2p^2 + m^2s^2). \qquad \text{[Eq. 8]}$$

The values of x and y for the extreme rivet are

$$x = \frac{(m-1)s}{2}, \qquad \text{[Eq. 9]}$$

and

$$y = \frac{(n-1)p}{2}; \qquad \text{[Eq. 10]}$$

and the total number of rivets in the group, N, is

$$N = nm. \qquad \text{[Eq. 11]}$$

These values of N, I, x, and y, when substituted in the equations given previously for S_d, S_y, and S_x, reduce them to the following forms:

$$S_d = \frac{P}{nm}, \qquad \text{[Eq. 12]}$$

$$S_y = \frac{6\,M\,(m-1)\,s}{nm\left[(n^2-1)\,p^2 + (m^2-1)\,s^2\right]}, \qquad \text{[Eq. 13]}$$

and

$$S_x = \frac{6\,M\,(n-1)\,p}{nm\left[(n^2-1)\,p^2 + (m^2-1)\,s^2\right]}. \qquad \text{[Eq. 14]}$$

The total stress S in any rivet will then be given by the formula,

$$S = \sqrt{(S_d + S_y)^2 + S_x^2}. \qquad \text{[Eq. 15]}$$

In any of the above equations, M may be replaced by Pe, if convenient.

When $m = 1$, these equations reduce to the forms,

$$S_d = \frac{P}{n}, \qquad \text{[Eq. 16]}$$

$$S_y = 0, \qquad \text{[Eq. 17]}$$

and

$$S_x = \frac{6\,M}{n\,(n+1)\,p} = \frac{6\,Pe}{n\,(n+1)\,p}. \qquad \text{[Eq. 18]}$$

When $P = 0$, $S = S_x$. When P is not zero, we may assume

$$\frac{e}{(n+1)\,p} = K, \qquad \text{[Eq. 19]}$$

whence we get for the value of S_x,

$$S_x = \frac{6\,PK}{n}, \qquad \text{[Eq. 20]}$$

and for the value of S,

$$S = \frac{P}{n}\sqrt{1 + 36\,K^2}. \qquad \text{[Eq. 21]}$$

When m is greater than unity, but still small as compared with n, it will be sufficiently accurate to use the formula,

$$S = \frac{P}{nm}\sqrt{1 + 36\,K^2}. \qquad \text{[Eq. 22]}$$

This equation errs a few per cent on the side of danger when K is less than 0.5; but for larger values of K the results are either correct or on the side of safety.

When $n = 1$, the equations for S_d, S_y, and S_x reduce to the forms,

$$S_d = \frac{P}{m}, \qquad \text{[Eq. 23]}$$

$$S_y = \frac{6\,M}{m\,(m+1)\,s}, \qquad \text{[Eq. 24]}$$

and

$$S_x = 0. \qquad \text{[Eq. 25]}$$

When $P = 0$, $S = S_y$. When P is not zero, we have for the value of S,

$$S = \frac{P}{m} + \frac{6\,M}{m\,(m+1)\,s} = \frac{P}{m}\left(1 + \frac{6\,e}{(m+1)\,s}\right). \qquad \text{[Eq. 26]}$$

If we assume,

$$\frac{e}{(m+1)s} = C, \qquad \text{[Eq. 27]}$$

Equation 26 becomes

$$S = \frac{P}{m}(1 + 6\,C). \qquad \text{[Eq. 28]}$$

If n be greater than unity, but still small as compared with m, it will be sufficiently accurate to write

$$S = \frac{P}{nm}(1 + 6\,C). \qquad \text{[Eq. 29]}$$

This equation errs on the safe side for all values of C.

Occasionally a group of rivets under eccentric load is forced to rotate about a point near the top or bottom of the said group, rather than at its mid-point. In such a case the moment of inertia of the group should be taken about this point of rotation, the distances r in Equations 2 and

3 being likewise measured thereform. If the rivets are evenly spaced, and $m = 1$, the moment of inertia of the group about a point located at a distance $\frac{p}{2}$ beyond the end rivet is $\frac{n}{3}(n^2 - \frac{1}{4})p^2$; and the distance of the extreme rivet from this same point is $(n - \frac{1}{2})\,p$. Substituting these values in Eq. 2, we have, since $S_x = S_b$,

$$S_x = \frac{3\,M\,(n - \frac{1}{2})\,p}{n\,(n^2 - \frac{1}{4})\,p^2} = \frac{3\,M}{n\,(n + \frac{1}{2})\,p}, \qquad \text{[Eq. 30]}$$

$$= \frac{3\,M}{n^2\,p} \text{ approximately.} \qquad \text{[Eq. 31]}$$

The last equation is sufficiently accurate for all purposes. It evidently errs slightly on the side of safety. The total stress in the extreme rivet is then given by the formula,

$$S = \frac{P}{n}\sqrt{1 + \frac{9\,M^2}{n^4\,p^2}}. \qquad \text{[Eq. 32]}$$

A group of rivets is frequently acted upon by a moment perpendicular to its plane, the effect thereof being to produce tension in certain of the rivets. In such a case the Y-axis can be taken in the plane of the rivet group perpendicular to the axis of the moment, and the X-axis perpendicular to the plane of the group. The value of S_x is given by Equation 6. S_y is zero, because x is zero for all of the rivets. For an even spacing of rivets, either Equation 18 or Equation 31 can be employed, the latter being the one usually applicable. If there are several rows of rivets, it will be necessary to divide the results given by the above equations by the number of rows. If the moment under consideration be due to a force which has a component perpendicular to the plane of the group, the stresses produced thereby in the rivets can be found by means of Equation 1 or Equation 16.

If a rivet group be subjected to forces which produce moments both in the plane of the group and perpendicular thereto, the two effects are to be computed separately. Moments in the plane of the group will produce shearing stresses in the various rivets, and those perpendicular to the said plane will produce tensile stresses. Forces acting parallel to the plane of the rivets cause shearing stresses, and those acting at right angles thereto cause tension or release of tension.

Another detail that should receive the careful attention of the designer is that of web splices in plate girders which are often figured for vertical shear only. In such a case they are likely to be very weak in certain respects. The correct method of designing such splices is explained fully in Chapter XXI.

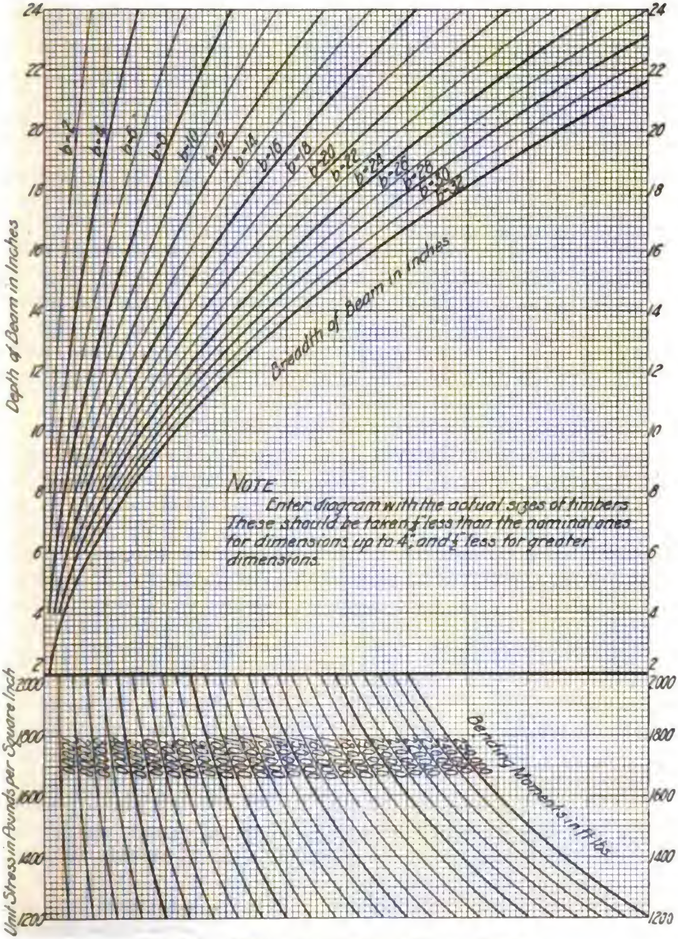
The lack of stiffeners at ends of I-beam spans (omitted because the web theoretically is strong enough without them) prevents a proper distribution of load over both the web and the masonry.

Failure to consider the longitudinal shear in ties and other wooden beams and the tendency to crush the timber across the grain at bearings often involves faulty detailing. In such members as ties, the actual unit shear will frequently be much less than that given by the theoretically correct formula $v = \frac{1.5\,V}{b\,d}$, in which v is the maximum unit shear—either horizontal or vertical,—V the total shear, b the width of the beam, and d its depth. For one thing, the rail may be so close to the stringer that shearing action can hardly develop; and if the rail base extends past the edge of the stringer flange, the correct value of V will be less than the load from the rail. Also, a tie always extends some distance past the centre of a stringer, usually a foot or more; and this portion of the timber will be subjected to horizontal shearing stresses, which will act to relieve those in the portion between the rail and the stringer. For instance, suppose that we have under consideration an 8″ × 10″ tie (nominal dimensions), for which the distance from the centre of the rail to the centre line of stringer is 12″ and the end overhang is 1′ – 6″, the shear in the portion between the rail and the stringer being 17,000 pounds. The maximum unit shear would be, theoretically, $\frac{17{,}000 \times 1.5}{7.5 \times 9.5}$, or 360 pounds per square inch. This assumes that the effective length for resisting this shear is the distance from the centre line of rail to the centre line of stringer, or 12″; but in reality this effective length is 12″ plus a certain amount of the 18″ overhang. The proportion of this overhang that should be counted in is uncertain, being practically the entire amount for short overhangs, and gradually reducing in percentage thereof as the length of the projection increases. For the case in hand 12″, or two-thirds of the overhang, will probably be a fair value; and under this assumption we find the unit shear to be $\frac{17{,}000 \times 1.5}{7.5 \times 9.5} \times \frac{12}{12 + 12}$, cr 180 pounds per square inch. This figure must, of course, be considered approximate.

In determining the rivet spacing in the loaded flanges of railway stringers and girders, the ignoring of the vertical shear on the rivets often causes them to be seriously overloaded. The correct method of computing the rivets under these conditions is given in Chapter XXI.

Fillers under ends of stringers should not be put in loose, but should be attached by two outside lines of rivets. The loose plates prevent the rivets from acting effectively and tend to overstress them in bending.

In figuring the size of any timber beam to resist a given bending moment, Fig. 16*h* will obviate the necessity of making any computations. This diagram was prepared by using the actual dimensions of the timbers, hence in entering it the actual or net sizes should be employed rather than the nominal ones. The difference between these is one-quarter (¼) inch up to all dimensions of four (4) inches, three-eighths (⅜) of an inch for those between four (4) and six (6) inches, and one-half (½) inch for those exceeding six (6) inches. To show the application of the double set of curves, let us assume that a timber beam capable of sustaining



NOTE
Enter diagram with the actual sizes of timbers. These should be taken ¼" less than the nominal ones for dimensions up to 4", and ⅜" less for greater dimensions.

Fig. 16h. Diagram for Designing Timber Beams.

properly an intensity of 1,600 pounds on the extreme fibre has to resist a bending moment from live load, impact, and dead load equal to 100,000 foot-pounds.

From the lower right-hand corner on the 1,600 horizontal line pass horizontally to the left until the 100,000 curve is reached; and from the point of intersection thereof pass vertically upward to any desired width-curve, and then horizontally to the left where the reading at the border of the sheet will give the net depth. If a 10″ gross width be assumed, a 9½″ net width will require a 22″ net depth or a 22½″ gross depth, or a timber 10″ × 24″, which is usually not a commercial section. Let us try a 12″ gross width, making the net width 11½″. This will give a net depth of 20″ or a gross depth of 20½″, requiring a 12″ × 22″ timber, which would be a good section, if the 22″ timbers are procurable. If not, we should try a 14″ gross width, which would give 18¼″ net depth or 18¾″ gross depth, necessitating a 14″ × 20″ stick. If this be too deep for commercial reasons, we should try a 16″ width and employ two sticks; hence the moment would be 50,000 foot-pounds per stick, requiring for a net width of 7½″ a net depth of 17½″ or a gross depth of 18″, hence two 8″ × 18″ timbers would be required. This section in most cases would afford the best solution of the problem.

The shoe of a girder or truss or the base-plate of a column is frequently subjected to an overturning moment as well as to a direct load, in which case it becomes necessary to compute the maximum pressures on the masonry, and frequently also the stresses in the anchor-bolts. If the moment is so small that anchor-bolts are theoretically unnecessary, the problem is quite simple; but if the bolts are under tension, an exact analysis is impossible. In an article by R. Fleming, Esq., C.E., in *Engineering News* of April 30, 1914, there are presented six different methods of calculating the stresses in anchor-bolts of chimneys and stacks, which give widely varying results. The editorial discussion that accompanies the article is especially valuable, as it points out that the stresses will depend largely upon such practical considerations as the rigidity of the shoe or base-plate, the deformation of the masonry, the stretch of the anchor bolts, and the amount which the anchor bolt nuts have been tightened. As all of these factors are more or less indeterminate, it is evident that an approximate solution only of the problem is possible. However, certain assumptions can be made which will give results that are sufficiently exact. An analysis that will apply to ordinary shoes and base-plates is herewith presented.

Suppose the base-plate shown in Fig. 16*i* to be acted upon by a central force P and a moment M; or, what amounts to the same thing, by a force P with an eccentricity e equal to $\frac{M}{P}$. Suppose further that the base-plate be assumed to remain straight. There are then five cases to be considered.

Case I. P acting downward, and $e < \frac{h}{6}$.

In this case there is no uplift at any point of the base-plate. The maximum intensity of pressure on the masonry, f_c, is given by the well-known formula,

$$f_c = \frac{P}{bh} + \frac{6M}{bh^2} = \frac{P}{bh}\left(1 + 6\frac{e}{h}\right). \qquad \text{[Eq. 33]}$$

Anchor bolts are theoretically unnecessary; and if they are used to provide an additional factor of safety against overturning, the unit tension therein is certain to be very low.

Case II. P acting downward, and $e > \frac{h}{6}$, but $< \frac{2h - a}{6}$

In this case the base-plate tends to lift from the masonry over a part or all of the space between the bolts on the windward side of the shoe and the adjacent edge thereof. It will be on the side of safety to assume the tension in these anchor-bolts to be zero, whence we find for the value of f_c the expression,

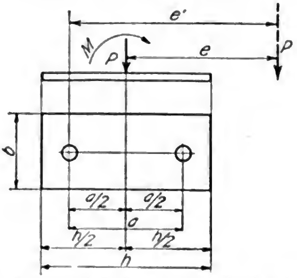


Fig. 16*i*. Layout of Base-Plate and Anchor Bolts.

$$f_c = \frac{2P}{3b\left(\frac{h}{2} - e\right)}. \qquad \text{[Eq. 34]}$$

As in Case I, there is no need of figuring the stresses in the anchor-bolts.

Case III. P acting downward, and $e > \frac{2h - a}{6}$.

In this case the shoe will tend to lift from the masonry at the bolts near the windward-edge, and these bolts will come into play unless the nuts thereof have been left loose, which condition can exist only through carelessness. If we assume that the nuts just begin to bear on the base-plate at the instant when the masonry pressure at the point where the bolts are located reduces to zero. the conditions will be analogous to those existing in a reinforced concrete beam under combined bending and direct longitudinal stress, with reinforcement on the tension side only. Fig. 37*m* gives curves for the calculation of stresses in such beams. This diagram could be applied directly if we could assume that the nuts were just at the elevation of the bottom of the base-plate, and that the unit stresses in both the concrete and the bolts either remain constant for some distance below the bottom of the base-plate, or else decrease in the same ratio; for Equation 93 of Chapter XXXVII is based upon these assumptions. It will be found, however, that the unit compression in the concrete will reduce very rapidly as we go downward, while the unit tension in the bolts will decrease more slowly. This condition will have the same effect upon the position of the neutral axis as though the width of the beam (which we may call b') were considerably greater than the width of the base-plate b, and as though the unit pressure in the concrete (which we may call f'_c) were likewise less than the unit pressure f_c under the base-plate, the ratio $\frac{f_c}{f'_c}$ being equal to $\frac{b'}{b}$. We may, therefore, utilize Fig. 37*m* by entering it with a steel ratio p given by the formula,

$$p = \frac{A_s}{b'd'} \qquad \text{[Eq. 35]}$$

(in which A_s is the total area of the anchor bolts near one edge of the base-plate), rather than with a ratio based upon the width b. The distance $e'\left(= e + \frac{a}{2}\right)$ is next figured, and then the ratio $\frac{d}{e'}$ is found. The value of R is computed by the formula,

$$R = \frac{Pe'}{b'd^2}. \qquad \text{[Eq. 36]}$$

We then enter the diagram on the upper left-hand margin with the value of $\frac{d}{e'}$, trace horizontally over to the curve for the value of p, and from this intersection trace vertically down to the value of R (found by tracing across from the lower left-hand margin). The values of f'_c and f_s at this last intersection point are then read by means of the two sets of diagonal lines, and the value of f_c is figured by the formula,

$$f_c = f'_c \frac{b'}{b}. \qquad \text{[Eq. 37]}$$

The correct value of the ratio $\frac{b'}{b}$ is unknown, but for ordinary cases it will probably lie somewhere between 1.5 and 2, the former value being the better when the bolts are embedded for nearly their full length, and the latter when a considerable portion thereof projects above the concrete. It will be found that a considerable change in this ratio will not affect materially the values of f_c and f_s. For instance, in the illustrative problem given below, $\frac{b'}{b}$ was assumed as 2, and the resulting values of f_c and f_s were 550 and 2700. If $\frac{b'}{b}$ had been taken as 1.5, we should have had $p = 0.44\%$, $R = 88$, $f'_c = 350$, $f_c = 530$, and $f_s = 3000$. If $\frac{b'}{b}$ had been assumed to be 2.5, we should have had $p = 0.26\%$, $R = 53$, $f'_c = 225$, $f_c = 560$, and $f_s = 2400$. If $\frac{b'}{b}$ had been taken as unity, we should have had $p = 0.66\%$, $R = 132$, $f'_c = f_c = 500$, and $f_s = 3400$.

The area A_s should be the gross area of the bolts, so that the value of f_s obtained in the above manner is that on the gross section. If the bolts are not upset, the unit stress at the root of the thread can be easily computed from the value of f_s.

Case IV. P acting upward, and $e < \frac{a}{2}$.

In this case we can assume the anchor bolts to take the entire load, the maximum stress T in the bolt or bolts in one side of the base-plate being given by the formula,

$$T = \frac{P}{2}\left(1 + \frac{2\,e}{a}\right). \qquad \text{[Eq. 38]}$$

Case V. P acting upward, and $e > \frac{a}{2}$.

In this case there will be tension on the bolt or bolts in one side of the base-plate, and compression on the masonry along the opposite edge thereof. This case can be solved in the same manner as Case III, using the dotted curves for p in Fig. 37*m*, and reading the values of $\frac{d}{e'}$ from the figures given in parentheses along the left-hand margin. The value of e' is negative in this case. This method gives the unit stresses in both the concrete and the anchor bolts.

The stresses in the anchor bolts can also be found by the following approximate method. It will be noticed that the compression area will hardly ever extend over a width as great as three-tenths of d (k being usually less than 0.3), so that it will be on the side of safety to assume that the compressive forces are concentrated at a point located at a distance $\frac{d}{10}$ from the edge of the base-plate. The tension in the anchor bolt or bolts can then be found by taking moments about this assumed centre of the compressive forçes, and dividing by $\frac{9}{10}\,d$. This gives us, for the stress T in the bolt or bolts,

$$T = \frac{P(e' + 0.9\,d)}{0.9\,d}. \qquad \text{[Eq. 39]}$$

To illustrate the application of the above methods, let us assume a base for which

$$b = 60'',$$
$$h = 50'',$$
$$a = 40'',$$

and $A_s = 18$ sq. in.,

and design it for five different conditions of loading, so chosen as to represent each of the five cases above discussed. We shall assume the anchor bolts to extend some distance above the base-plates, and to be upset.

Case I.

$$P = 1{,}200{,}000 \text{ lbs. (acting downward).}$$
$$M = 8{,}000{,}000 \text{ in.-lbs.}$$
$$\therefore e = 6.7''.$$
$$\tfrac{1}{6}\,h = 8.33''. \quad \therefore e < \frac{h}{6}.$$
$$\therefore f_c = \frac{1{,}200{,}000}{60 \times 50}\left(1 + \frac{6 \times 6.7}{50}\right),$$
$$= 400\,(1 + 0.8) = 720 \text{ lbs. per sq. in.}$$

Case II.

$$P = 850{,}000 \text{ lbs. (acting downward).}$$
$$M = 8{,}000{,}000 \text{ in.-lbs.}$$
$$\therefore e = 9.4''.$$
$$\tfrac{1}{6}\,h = 8.33''.$$
$$\frac{2\,h - a}{6} = \frac{100 - 40}{6} = 10''. \quad \therefore e > \frac{h}{6} \text{ and } < \frac{2\,h - a}{6}.$$
$$\therefore f_c = \frac{2 \times 850{,}000}{3 \times 60\,(25 - 9.4)} = 605 \text{ lbs. per sq. in.}$$

Case III.

$$P = 400{,}000 \text{ lbs. (acting downward).}$$
$$M = 8{,}000{,}000 \text{ in.-lbs.}$$
$$\therefore e = 20''.$$
$$\frac{2\,h - a}{6} = \frac{100 - 40}{6} = 10''. \quad \therefore e > \frac{2\,h - a}{6}.$$
$$d = 20 + 25 = 45.$$
$$e' = 20 + 20 = 40.$$
$$\therefore \frac{d}{e'} = \frac{45}{40} = 1.125.$$
$$\text{Assume } \frac{b'}{b} = 2. \quad \therefore b' = 2 \times 60 = 120.$$

$$p' = \frac{18}{120 \times 45} = 0.33\%.$$

$$R = \frac{400{,}000 \times 40}{120 \times (45)^2} = 66.$$

$$\therefore f'_c = 275 \text{ lbs. per sq. in.,}$$

$$f_c = 2 \times 275 = 550 \text{ lbs. per sq. in.,}$$

and $$f_s = 2{,}700 \text{ lbs. per sq. in.}$$

Case IV.

$$P = 300{,}000 \text{ lbs. (acting upward).}$$

$$M = 3{,}000{,}000 \text{ in.-lbs.}$$

$$\therefore e = 10''.$$

$$a/2 = 20''. \quad \therefore e < \frac{a}{2}.$$

$$\therefore f_s = \frac{300{,}000}{2 \times 18}\left(1 + \frac{20}{40}\right) = 12{,}500 \text{ lbs. per sq. in.}$$

Case V.

$$P = 160{,}000 \text{ lbs. (acting upward).}$$

$$M = 8{,}000{,}000 \text{ in.-lbs.}$$

$$\therefore e = 50''.$$

$$\frac{a}{2} = 20''. \quad \therefore e > \frac{a}{2}.$$

Exact Method.

$$d = 20 + 25 = 45.$$

$$e' = 20 - 50 = -30.$$

$$\frac{d}{e'} = \frac{45}{-30} = -1.5.$$

Assume $$\frac{b'}{b} = 2. \quad \therefore b' = 2 \times 60 = 120.$$

$$p' = \frac{18}{120 \times 45} = 0.33\%.$$

$$R = \frac{160{,}000 \times 30}{120 \times (45)^2} = 19.7.$$

$$\therefore f'_c = 230 \text{ lbs. per sq. in.,}$$

$$f_c = 2 \times 230 = 460 \text{ lbs. per sq. in.,}$$

and $$f_s = 15{,}000 \text{ lbs. per sq. in. nearly.}$$

Approximate Method.

$$d = 20 + 25 = 45.$$

$$e' = 20 - 50 = -30.$$

$$\therefore T = \frac{160{,}000\ (30 + 0.9 \times 45)}{0.9 \times 45} = 279{,}000 \text{ lbs.}$$

and $$f_s = \frac{279{,}000}{18} = 15{,}500 \text{ lbs. per sq. in.}$$

To facilitate further the work of detailing, a number of tables which at various times the author has prepared, and which he has found quite useful in his practice, will now be given.

In Tables 16*a* to 16*d* inclusive, there is presented much information of value concerning wire ropes and their details. Table 16*a* gives the weight per lineal foot, area, ultimate strength, and elastic limit for plow steel rope of six strands of nineteen wires each and a hemp centre, varying from one-quarter inch to three inches in diameter. Table 16*b* shows the bending stresses on these ropes over sheaves varying in diameter from ten to two hundred and sixteen inches. For each rope there are given values for all the sheave diameters over which they are likely to run. Table 16*c* indicates for open sockets the various dimensions thereof for connecting the ropes of different diameters. Table 16*d* gives like information for closed sockets.

TABLE 16*a*

PROPERTIES OF 6 × 19 PLOW STEEL WIRE ROPES.

Rope Diameter (Inches)	Weight per Lineal Foot (Pounds)	Area (Square Inches)	Ultimate Strength (Pounds)	Elastic Limit (Pounds)
¼	.10	.02744	6,000	3,000
⅜	.22	.06261	12,000	7,000
½	.39	.10748	21,000	13,000
⅝	.62	.16590	34,000	20,000
¾	.89	.22886	47,000	28,000
⅞	1.20	.31412	63,000	37,000
1	1.58	.40525	81,000	48,000
1⅛	2.00	.50467	101,000	61,000
1¼	2.45	.62550	124,000	74,000
1⅜	3.00	.75300	151,000	90,000
1½	3.55	.90912	176,000	106,000
1⅝	4.15	1.07168	198,000	119,000
1¾	4.85	1.25383	239,000	144,000
1⅞	5.55	1.39000	270,000	162,000
2	6.30	1.61274	298,000	179,000
2¼	8.00	2.07479	378,000	227,000
2½	9.85	2.48404	474,000	284,000
2¾	11.95	3.05205	567,000	340,000
3	14.22	3.63732	677,000	406,000

In Table 16*e* will be found the pitch diameters for gears with circular pitches, varying from one-eighth of an inch to six inches and having a total number of teeth varying from twelve to ninety-nine.

Tables 16*f* and 16*g* record the intensities of working stresses for compression members of carbon steel with both fixed ends and hinged ends.

Table 16*h* gives the intensities of working stresses for forked ends and extension-plates of compression members built of carbon steel.

Shearing and bearing values of carbon steel rivets are recorded in Table 16*i*.

TABLE 16b

BENDING STRESSES IN 6 × 19 WIRE ROPES.

Sheave Diameter, Inches	Rope Diameter										
	¼"	⅜"	½"	⅝"	¾"	⅞"	1"	1⅛"	1¼"	1⅜"	1½"
10	1,120	.....	.....	......	......	......	......	......	......	......	......
12	940	3,130	7,350	......	......	......	......	......	......	......	......
15	750	2,520	5,920	11,480	......	......	......	......	......	......	......
18	630	2,110	4,960	9,630	16,520	......	......	......	......	......	......
20	570	1,900	4,480	8,710	14,950	......	......	......	......	......	......
24	470	1,590	3,750	7,280	12,550	......	......	......	......	......	......
30	.....	1,280	3,000	5,850	10,070	15,900	......	......	......	......	......
36	.....	.....	2,510	4,900	8,450	13,320	......	......	......	......	......
40	.....	.....	.....	4,410	7,620	12,050	17,900	......	......	......	......
42	.....	.....	.....	4,200	7,250	11,500	17,100	24,200	......	......	......
46	.....	.....	.....	......	6,630	10,500	15,620	22,250	......	......	......
48	.....	.....	.....	......	6,360	10,060	14,960	21,300	......	......	......
50	.....	.....	.....	......	......	9,720	14,400	20,400	28,100	......	......
56	.....	.....	.....	......	......	8,650	12,800	18,300	25,600	33,380	......
60	.....	.....	.....	......	......	8,070	12,050	17,100	23,500	31,200	40,284
66	.....	.....	.....	......	......	7,380	11,000	15,600	21,300	28,400	36,700
72	.....	.....	.....	......	......	6,750	10,050	14,300	19,500	26,000	33,700
76	.....	.....	.....	......	......	6,410	9,550	13,600	18,550	24,700	32,000
84	.....	.....	.....	......	......	5,800	8,690	12,300	16,850	22,400	29,000
96	.....	.....	.....	......	......	......	7,580	10,770	14,750	19,700	25,450
108	.....	.....	.....	......	......	......	......	9,580	13,110	17,450	22,650
120	.....	.....	.....	......	......	......	......	......	11,830	15,700	20,400
132	.....	.....	.....	......	......	......	......	......	......	14,300	18,520
144	.....	.....	.....	......	......	......	......	......	......	......	17,050

TABLE 16b (*Continued*)

BENDING STRESSES IN 6 × 19 WIRE ROPES.

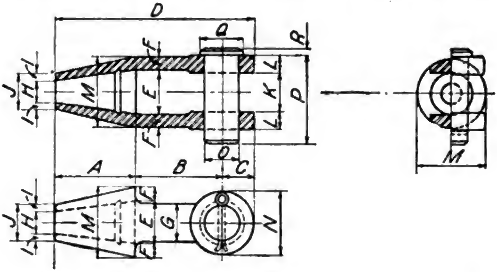
Sheave Diameter, Inches	Rope Diameter									
	1⅝"	1¾"	1⅞"	2"	2⅛"	2¼"	2⅜"	2½"	2¾"	3"
108	28,750	35,800	......	......	......	......	......	......	......	....
120	25,900	32,300	39,700	48,300	......	......	......	......	......	....
132	23,600	29,700	36,100	43,800	......	......	......	......	......	....
144	21,600	27,000	33,200	40,200	48,200	57,300	67,200	......	......	....
156	20,000	24,900	30,700	37,200	44,600	52,800	62,100	72,400	......	....
162	19,300	24,100	29,600	35,800	42,900	50,800	59,900	69,800	......	....
168	18,600	23,200	28,460	34,600	41,400	49,100	57,800	67,500	89,600	....
180	17,400	21,700	26,600	32,300	38,700	45,900	54,000	63,000	83,600	108,200
192	......	......	......	30,300	36,400	43,000	50,600	59,200	78,600	101,500
204	......	......	......	28,500	34,200	40,500	47,600	55,600	74,000	95,700
216	......	......	......	26,900	32,300	38,300	45,000	52,600	70,000	90,600

In Table 16*j* are given the bearing values of carbon steel pins; and in Table 16*k* are shown the allowable bending moments thereon for both carbon steel and nickel steel. The reason for inserting in the last-mentioned table a column giving the bending moments on pins of nickel steel, while no other tabulations for that alloy are recorded elsewhere in the book, is that nickel steel pins are occasionally employed in carbon steel bridges in order to reduce diameters which otherwise would be excessive. Until the steel manufacturers and the bridge shops jointly can determine finally upon standard specifications for the qualities of the several classes of nickel steel required in bridgework, it would be useless labor to compute a set of tables in order to facilitate the designing of nickel steel structures.

In the detailing of substructure the most common fault is bringing the bearing from the span too close to the edge of the masonry and thus crushing off pieces thereof. Another rather common fault consists in designing piers with pile foundation by putting an ice break on the upstream end and no corresponding offset at the lower end, with the result that there is an uneven distribution of load over the group of piles.

TABLE 16c

DETAILED DIMENSIONS FOR OPEN SOCKETS FOR WIRE ROPES.



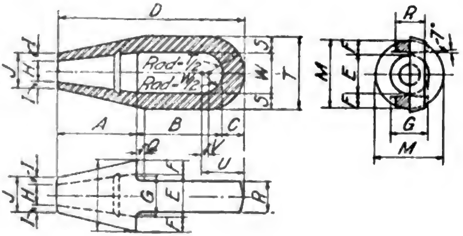
Diam. Rope	Detail Dimensions, in Inches																		Cotter		Weight Lbs.
	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	Dia.	Lgth	
1/4	2	1 9/16	3/4	4 5/16	13/16	1/4	3/4	5/16	1/8	9/16	11/16	5/16	1 5/16	1 5/16	11/16	1 3/4	7/8	3/16	3/16	1 1/8	.88
3/8	2	1 3/4	7/8	4 5/8	15/16	5/16	13/16	7/16	5/32	3/4	13/16	13/32	1 9/16	1 1/2	13/16	2 1/16	1	1/4	3/16	1 1/4	......
1/2	2 1/2	2	1 1/16	5 9/16	1 1/8	3/8	1	9/16	3/16	15/16	1	1/2	1 7/8	1 7/8	1	2 7/16	1 1/4	1/4	3/16	1 7/16	2.25
5/8	3	2 1/2	1 1/4	6 3/4	1 3/8	7/16	1 1/4	11/16	7/32	1 1/8	1 1/4	9/16	2 1/4	2 1/4	1 3/16	2 7/8	1 1/2	5/16	1/4	1 3/4	3.75
3/4	3 1/2	3	1 7/16	7 15/16	1 5/8	1/2	1 1/2	13/16	7/32	1 1/4	1 1/2	5/8	2 5/8	2 5/8	1 3/8	3 1/4	1 3/4	3/8	1/4	2	5.75
7/8	4	3 1/2	1 3/4	9 1/4	1 7/8	5/8	1 3/4	15/16	9/32	1 1/2	1 3/4	3/4	3 1/8	3 1/8	1 5/8	3 7/8	2	3/8	5/16	2 1/4	10
1	4 1/2	4	2 1/16	10 9/16	2 1/4	11/16	2	1 1/8	5/16	1 3/4	2	7/8	3 5/8	3 3/4	2	4 1/2	2 1/2	7/16	3/8	2 3/4	......
1 1/8	5	4 1/2	2 5/16	11 13/16	2 1/2	3/4	2 3/8	1 1/4	3/8	2	2 1/4	1	4	4 1/8	2 1/4	5	2 3/4	1/2	3/8	2 3/4	......
1 1/4	5 1/2	5	2 11/16	13 3/16	2 3/4	15/16	2 3/4	1 3/8	7/16	2 1/4	2 1/2	1 1/8	4 5/8	4 3/4	2 5/8	5 5/8	3 1/4	5/8	7/16	3 1/2	......
1 3/8	5 1/2	5	2 11/16	13 3/16	2 3/4	15/16	2 3/4	1 1/2	3/8	2 1/4	2 1/2	1 1/8	4 5/8	4 3/4	2 5/8	5 5/8	3 1/4	5/8	7/16	3 1/2	......
1 1/2	6 1/2	6 1/2	3 1/4	16 1/4	3 1/4	1 1/8	3 1/4	1 5/8	9/16	2 3/4	3	1 5/16	5 1/2	5 3/4	3	6 5/8	3 3/4	11/16	1/2	4	55
1 5/8	6 1/2	6 1/2	3 1/4	16 1/4	3 1/4	1 1/8	3 1/4	1 3/4	1/2	2 3/4	3	1 5/16	5 1/2	5 3/4	3	6 5/8	3 3/4	11/16	1/2	4	55
1 3/4	7 1/2	7	3 3/4	18 1/4	3 3/4	1 5/16	3 7/8	1 7/8	5/8	3 1/8	3 1/2	1 9/16	6 3/8	6 1/2	3 1/2	7 5/8	4 1/2	3/4	1/2	4 1/2	82
1 7/8	7 1/2	7	3 3/4	18 1/4	3 3/4	1 5/16	3 7/8	2	9/16	3 1/8	3 1/2	1 9/16	6 3/8	6 1/2	3 1/2	7 5/8	4 1/2	3/4	1/2	4 1/2	82
2	8 1/2	9	4	21 1/2	4 3/8	1 7/16	4 1/4	2 1/8	13/16	3 3/4	4	1 13/16	7 1/4	7	3 3/4	8 3/4	4 5/8	7/8	1/2	4 3/4	120
2 1/8	8 1/2	9	4	21 1/2	4 3/8	1 7/16	4 1/4	2 1/4	3/4	3 3/4	4	1 13/16	7 1/4	7	3 3/4	8 3/4	4 5/8	7/8	1/2	4 3/4	120
2 1/4	9	10	4 1/2	23 1/2	5	1 5/8	3 3/8	2 3/8	13/16	4	4 1/2	2 1/8	8 1/4	7 3/4	4 1/4	9 7/8	5	7/8	1/2	5 1/4	160
2 3/8	9	10	4 1/2	23 1/2	5	1 5/8	4 3/8	2 1/2	3/4	4	4 1/2	2 1/8	8 1/4	7 3/4	4 1/4	9 7/8	5	7/8	1/2	5 1/4	160

The subject of faulty detailing dwelt upon at such length in this chapter might be carried much farther, but it is not advisable so to extend it; because the examples given will generally be found sufficient to teach the designer "how not to do it," while a careful study of Chapter XV on "First Principles of Designing" will enable him to adopt methods of detailing that will not vary greatly from those which are ideally correct.

Should any one desire to study exhaustively the methods of correct designing of the details for ordinary railway and highway bridge superstructures, he can do no better than to read Prof. John E. Kirkham's excellent treatise on "Structural Engineering." While evidently written for the purpose of class-room instruction, it is well worthy of perusal by practising engineers.

TABLE 16*d*

Detailed Dimensions for Closed Sockets for Wire Ropes.



Diam. Rope	Detail Dimensions, in Inches																		Weight, Lbs.
	A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*M*	*Q*	*R*	*S*	*T*	*U*	*V*	*W*	
1/4	1 3/4	2	1/2	4 1/4	7/8	5/16	13/16	5/16	27/128	47/64	1 1/2	3/4	5/8	3/8	1 5/8	15/16	1/8	7/8	0.66
3/8	1 3/4	2	1/2	4 1/4	7/8	5/16	13/16	7/16	19/128	47/64	1 1/2	3/4	5/8	3/8	1 5/8	15/16	1/8	7/8	0.66
1/2	2	2 1/4	9/16	4 13/16	1 1/16	3/8	1	9/16	5/16	15/16	1 13/16	3/4	3/4	7/16	1 15/16	1 3/32	1/8	1 1/16	1.1
5/8	2 1/2	2 1/2	11/16	5 11/16	1 5/16	7/16	1 1/4	11/16	13/64	1 3/32	2 3/16	3/4	7/8	1/2	2 5/16	1 11/32	3/16	1 5/16	1.8
3/4	3	3	13/16	6 13/16	1 9/16	1/2	1 1/2	13/16	7/32	1 1/4	2 9/16	3/4	1 1/16	5/8	2 11/16	1 19/32	3/16	1 9/16	3.2
7/8	3 1/2	3 1/2	1	8	1 13/16	19/32	1 3/4	15/16	17/64	1 15/32	3	7/8	1 1/4	3/4	3 5/16	1 29/32	1/4	1 13/16	5
1	4	4	1 1/8	9 1/8	2 1/8	11/16	2	1 1/8	5/16	1 3/4	3 1/2	1	1 1/2	7/8	3 7/8	2 3/16	1/4	2 1/8	8
1 1/8	4 1/2	4 1/2	1 1/4	10 1/4	2 3/8	25/32	2 1/4	1 1/4	23/64	1 31/32	3 15/16	1 1/8	1 3/4	15/16	4 1/4	2 7/16	5/16	2 3/8	11 1/4
1 1/4	5	5	1 5/16	11 5/16	2 5/8	7/8	2 1/2	1 3/8	13/32	2 3/16	4 3/8	1 1/4	2	1	4 5/8	2 5/8	5/16	2 5/8	15 1/2
1 3/8	5	5	1 5/16	11 5/16	2 5/8	7/8	2 1/2	1 1/2	11/32	2 3/16	4 3/8	1 1/4	2	1	4 5/8	2 5/8	5/16	2 5/8	15 1/2
1 1/2	6	6 1/2	1 1/2	14	3 1/8	1 1/16	3	1 5/8	1/2	2 5/8	5 1/4	1 1/2	2 1/2	1 1/8	5 3/8	3 1/16	3/8	3 1/8	25 3/4
1 5/8	6	6 1/2	1 1/2	14	3 1/8	1 1/16	3	1 3/4	7/16	2 5/8	5 1/4	1 1/2	2 1/2	1 1/8	5 3/8	3 1/16	3/8	3 1/8	25 3/4
1 3/4	7 1/2	7 13/16	2 3/16	17 1/2	3 3/4	1 5/16	4	1 7/8	5/8	3 1/8	6 3/8	1 1/2	3	1 7/16	6 3/4	4	5/8	3 5/8	53
2	8 1/2	8 13/16	2 7/16	19 3/4	4 3/8	1 9/16	4 1/4	2 1/8	13/16	3 3/4	7 1/2	2 1/4	3 1/4	1 1/2	7 3/4	4 3/8	1/2	3 7/8	80
2 1/4	9	9 3/4	2 7/8	21 5/8	5	1 5/8	5 1/2	2 3/8	13/16	4	8 1/4	2 1/2	3 5/8	1 5/8	8 1/2	4 7/8	5/8	4	105

TABLE 16e

PITCH DIAMETERS OF GEARS FOR CIRCULAR PITCH.

No. of Teeth	Circular Pitch in Inches								No. of Teeth
	1/8	1/4	3/8	7/16	1/2	5/8	3/4	7/8	
12	.4775	.9549	1.432	1.671	1.910	2.387	2.865	3.342	12
13	.5173	1.035	1.552	1.810	2.069	2.586	3.104	3.621	13
14	.5570	1.114	1.671	1.950	2.228	2.785	3.342	3.899	14
15	.5968	1.194	1.790	2.089	2.387	2.984	3.581	4.178	15
16	.6366	1.273	1.910	2.228	2.546	3.183	3.820	4.456	16
17	.6764	1.353	2.029	2.367	2.706	3.382	4.058	4.735	17
18	.7162	1.432	2.149	2.507	2.865	3.581	4.297	5.013	18
19	.7560	1.512	2.268	2.646	3.024	3.780	4.536	5.292	19
20	.7958	1.592	2.387	2.785	3.183	3.979	4.775	5.570	20
21	.8356	1.671	2.507	2.924	3.342	4.178	5.013	5.849	21
22	.8754	1.751	2.626	3.064	3.501	4.377	5.252	6.127	22
23	.9151	1.830	2.745	3.203	3.661	4.576	5.491	6.406	23
24	.9549	1.910	2.865	3.342	3.820	4.775	5.730	6.685	24
25	.9947	1.989	2.984	3.482	3.979	4.974	5.968	6.963	25
26	1.035	2.069	3.104	3.621	4.138	5.173	6.207	7.242	26
27	1.074	2.149	3.223	3.760	4.297	5.371	6.446	7.520	27
28	1.114	2.228	3.342	3.899	4.456	5.570	6.685	7.799	28
29	1.154	2.308	3.462	4.039	4.615	5.769	6.923	8.077	29
30	1.194	2.387	3.581	4.178	4.775	5.968	7.162	8.356	30
31	1.233	2.467	3.700	4.317	4.934	6.167	7.401	8.634	31
32	1.273	2.546	3.820	4.456	5.093	6.366	7.639	8.913	32
33	1.313	2.626	3.939	4.596	5.252	6.565	7.878	9.191	33
34	1.353	2.706	4.058	4.735	5.411	6.764	8.117	9.470	34
35	1.393	2.785	4.178	4.874	5.570	6.963	8.356	9.748	35
36	1.432	2.865	4.297	5.013	5.730	7.162	8.594	10.027	36
37	1.472	2.944	4.417	5.153	5.889	7.361	8.833	10.305	37
38	1.512	3.024	4.536	5.292	6.048	7.560	9.072	10.584	38
39	1.552	3.104	4.655	5.431	6.207	7.759	9.311	10.862	39
40	1.592	3.183	4.775	5.570	6.366	7.958	9.549	11.141	40
41	1.631	3.263	4.894	5.710	6.525	8.157	9.788	11.419	41
42	1.671	3.342	5.013	5.849	6.685	8.356	10.027	11.698	42
43	1.711	3.422	5.133	5.988	6.844	8.555	10.265	11.976	43
44	1.751	3.501	5.252	6.127	7.003	8.754	10.504	12.255	44
45	1.790	3.581	5.371	6.267	7.162	8.952	10.743	12.533	45
46	1.830	3.661	5.491	6.406	7.321	9.151	10.982	12.812	46
47	1.870	3.740	5.610	6.545	7.480	9.350	11.220	13.090	47
48	1.910	3.820	5.730	6.685	7.639	9.549	11.459	13.369	48
49	1.950	3.899	5.849	6.824	7.799	9.748	11.698	13.648	49
50	1.989	3.979	5.968	6.963	7.958	9.947	11.937	13.926	50
51	2.029	4.058	6.088	7.102	8.117	10.146	12.175	14.205	51
52	2.069	4.138	6.207	7.242	8.276	10.345	12.414	14.483	52
53	2.109	4.218	6.326	7.381	8.435	10.544	12.653	14.762	53
54	2.149	4.297	6.446	7.520	8.594	10.743	12.892	15.040	54

Outside diameter of any gear is equal to the pitch diameter of a similar gear with two additional teeth.

Inside diameter of any gear is equal to the pitch diameter of a similar gear with two less teeth, minus the clearance.

These two rules do not hold for gears with less than 17 teeth.

TABLE 16e (*Continued*)

Pitch Diameters of Gears for Circular Pitch.

No. of Teeth	Circular Pitch in Inches								No. of Teeth
	1/8	1/4	3/8	7/16	1/2	5/8	3/4	7/8	
55	2.188	4.377	6.565	7.659	8.754	10.942	13.130	15.319	55
56	2.228	4.456	6.685	7.799	8.913	11.141	13.369	15.597	56
57	2.268	4.536	6.804	7.937	9.072	11.340	13.608	15.876	57
58	2.308	4.615	6.923	8.077	9.231	11.539	13.846	16.154	58
59	2.348	4.695	7.043	8.216	9.390	11.738	14.085	16.433	59
60	2.387	4.775	7.162	8.356	9.549	11.937	14.324	16.711	60
61	2.427	4.854	7.281	8.495	9.708	12.136	14.563	16.990	61
62	2.467	4.934	7.401	8.634	9.868	12.335	14.801	17.268	62
63	2.507	5.013	7.520	8.773	10.027	12.533	15.040	17.547	63
64	2.546	5.093	7.639	8.913	10.186	12.732	15.279	17.825	64
65	2.586	5.173	7.759	9.052	10.345	12.931	15.518	18.104	65
66	2.626	5.252	7.878	9.191	10.504	13.130	15.756	18.382	66
67	2.666	5.332	7.998	9.330	10.663	13.329	15.995	18.661	67
68	2.706	5.411	8.117	9.470	10.823	13.528	16.234	18.939	68
69	2.745	5.491	8.236	9.609	10.982	13.727	16.473	19.218	69
70	2.785	5.570	8.356	9.748	11.141	13.926	16.711	19.496	70
71	2.825	5.650	8.475	9.888	11.300	14.125	16.950	19.775	71
72	2.865	5.730	8.594	10.027	11.459	14.324	17.189	20.054	72
73	2.905	5.809	8.714	10.166	11.618	14.523	17.427	20.332	73
74	2.944	5.889	8.833	10.305	11.777	14.722	17.666	20.611	74
75	2.984	5.968	8.952	10.445	11.937	14.921	17.905	20.889	75
76	3.024	6.048	9.072	10.584	12.096	15.120	18.144	21.168	76
77	3.064	6.127	9.191	10.723	12.255	15.319	18.382	21.446	77
78	3.104	6.207	9.311	10.862	12.414	15.518	18.621	21.725	78
79	3.143	6.287	9.430	11.002	12.573	15.717	18.860	22.003	79
80	3.183	6.366	9.549	11.141	12.732	15.915	19.099	22.282	80
81	3.223	6.446	9.669	11.280	12.892	16.114	19.337	22.560	81
82	3.263	6.525	9.788	11.419	13.051	16.313	19.576	22.839	82
83	3.302	6.605	9.907	11.559	13.210	16.512	19.815	23.117	83
84	3.342	6.685	10.027	11.698	13.369	16.711	20.054	23.396	84
85	3.382	6.764	10.146	11.837	13.528	16.910	20.292	23.674	85
86	3.422	6.844	10.265	11.976	13.687	17.109	20.531	23.953	86
87	3.462	6.923	10.385	12.116	13.846	17.308	20.770	24.231	87
88	3.501	7.003	10.504	12.255	14.006	17.507	21.008	24.510	88
89	3.541	7.082	10.624	12.394	14.165	17.706	21.247	24.788	89
90	3.581	7.162	10.743	12.533	14.324	17.905	21.486	25.067	90
91	3.621	7.242	10.862	12.673	14.483	18.104	21.725	25.345	91
92	3.661	7.321	10.982	12.812	14.642	18.303	21.963	25.624	92
93	3.700	7.401	11.101	12.951	14.801	18.502	22.202	25.902	93
94	3.740	7.480	11.220	13.090	14.961	18.701	22.441	26.181	94
95	3.780	7.560	11.340	13.230	15.120	18.900	22.680	26.460	95
96	3.820	7.639	11.459	13.369	15.279	19.099	22.918	26.738	96
97	3.860	7.719	11.579	13.508	15.438	19.298	23.157	27.017	97
98	3.899	7.799	11.698	13.648	15.597	19.497	23.396	27.295	98
99	3.939	7.878	11.817	13.789	15.756	19.695	23.635	27.574	99

Outside diameter of any gear is equal to the pitch diameter of a similar gear with two additional teeth.

Inside diameter of any gear is equal to the pitch diameter of a similar gear with two less teeth, minus the clearance.

These two rules do not hold for gears with less than 17 teeth.

TABLE 16c (*Continued*)

PITCH DIAMETERS OF GEARS FOR CIRCULAR PITCH.

No. of Teeth	CIRCULAR PITCH IN INCHES							No. of Teeth
	1	1⅛	1¼	1⅜	1½	1¾	2	
12	3.820	4.297	4.775	5.252	5.730	6.685	7.639	12
13	4.138	4.655	5.173	5.690	6.207	7.242	8.276	13
14	4.456	5.013	5.570	6.127	6.685	7.799	8.913	14
15	4.775	5.371	5.968	6.565	7.162	8.356	9.549	15
16	5.093	5.730	6.366	7.003	7.639	8.913	10.186	16
17	5.411	6.088	6.764	7.440	8.117	9.470	10.823	17
18	5.730	6.446	7.162	7.878	8.594	10.027	11.459	18
19	6.048	6.804	7.560	8.316	9.072	10.584	12.096	19
20	6.366	7.162	7.958	8.754	9.549	11.141	12.732	20
21	6.685	7.520	8.356	9.191	10.027	11.698	13.369	21
22	7.003	7.878	8.754	9.629	10.504	12.255	14.006	22
23	7.321	8.236	9.151	10.067	10.982	12.812	14.642	23
24	7.639	8.594	9.549	10.504	11.459	13.369	15.279	24
25	7.958	8.952	9.947	10.942	11.937	13.926	15.915	25
26	8.276	9.311	10.345	11.380	12.414	14.483	16.552	26
27	8.594	9.669	10.743	11.817	12.892	15.040	17.189	27
28	8.913	10.027	11.141	12.255	13.369	15.597	17.825	28
29	9.231	10.385	11.539	12.693	13.846	16.154	18.462	29
30	9.549	10.743	11.937	13.130	14.324	16.711	19.099	30
31	9.868	11.101	12.335	13.568	14.801	17.268	19.735	31
32	10.186	11.459	12.732	14.006	15.279	17.825	20.372	32
33	10.504	11.817	13.130	14.443	15.756	18.382	21.008	33
34	10.823	12.175	13.528	14.881	16.234	18.939	21.645	34
35	11.141	12.533	13.926	15.319	16.711	19.496	22.282	35
36	11.459	12.892	14.324	15.756	17.189	20.054	22.918	36
37	11.777	13.250	14.722	16.194	17.666	20.611	23.555	37
38	12.096	13.608	15.120	16.632	18.144	21.168	24.192	38
39	12.414	13.966	15.518	17.069	18.621	21.725	24.828	39
40	12.732	14.324	15.915	17.507	19.099	22.282	25.465	40
41	13.051	14.682	16.313	17.945	19.576	22.839	26.101	41
42	13.369	15.040	16.711	18.382	20.054	23.396	26.738	42
43	13.687	15.398	17.109	18.820	20.531	23.953	27.375	43
44	14.006	15.756	17.507	19.258	21.008	24.510	28.011	44
45	14.324	16.114	17.905	19.695	21.486	25.067	28.648	45
46	14.642	16.473	18.303	20.133	21.963	25.624	29.285	46
47	14.961	16.831	18.701	20.571	22.441	26.181	29.921	47
48	15.279	17.189	19.099	21.008	22.918	26.738	30.558	48
49	15.597	17.547	19.496	21.446	23.396	27.295	31.194	49
50	15.915	17.905	19.894	21.884	23.873	27.852	31.831	50
51	16.234	18.263	20.292	22.321	24.351	28.409	32.468	51
52	16.552	18.621	20.690	22.759	24.828	28.966	33.104	52
53	16.870	18.979	21.089	23.197	25.306	29.523	33.741	53
54	17.189	19.337	21.486	23.635	25.783	30.080	34.377	54

Outside diameter of any gear is equal to the pitch diameter of a similar gear with two additional teeth.

Inside diameter of any gear is equal to the pitch diameter of a similar gear with two less teeth, minus the clearance.

These two rules do not hold for gears with less than 17 teeth.

TABLE 16e (*Continued*)

PITCH DIAMETERS OF GEARS FOR CIRCULAR PITCH.

No. of Teeth	CIRCULAR PITCH IN INCHES							No. of Teeth
	1	1⅛	1¼	1⅜	1½	1¾	2	
55	17.507	19.695	21.884	24.072	26.261	30.637	35.014	55
56	17.825	20.054	22.282	24.510	26.738	31.194	35.651	56
57	18.144	20.412	22.680	24.948	27.215	31.751	36.287	57
58	18.462	20.770	23.077	25.385	27.693	32.308	36.924	58
59	18.780	21.128	23.475	25.823	28.170	32.865	37.561	59
60	19.099	21.486	23.873	26.261	28.648	33.423	38.197	60
61	19.417	21.844	24.271	26.698	29.125	33.980	38.834	61
62	19.735	22.202	24.669	27.136	29.603	34.537	39.470	62
63	20.054	22.560	25.067	27.574	30.080	35.094	40.107	63
64	20.372	22.918	25.465	28.011	30.558	35.651	40.744	64
65	20.690	23.276	25.863	28.449	31.035	36.208	41.380	65
66	21.008	23.635	26.261	28.887	31.513	36.765	42.017	66
67	21.327	23.993	26.658	29.324	31.990	37.322	42.654	67
68	21.645	24.351	27.056	29.762	32.468	37.879	43.290	68
69	21.963	24.709	27.454	30.200	32.945	38.436	43.927	69
70	22.282	25.067	27.852	30.637	33.423	38.993	44.563	70
71	22.600	25.425	28.250	31.075	33.900	39.550	45.200	71
72	22.918	25.783	28.648	31.513	34.377	40.107	45.837	72
73	23.237	26.141	29.046	31.950	34.855	40.664	46.473	73
74	23.555	26.499	29.444	32.388	35.332	41.221	47.110	74
75	23.873	26.857	29.842	32.826	35.810	41.778	47.746	75
76	24.192	27.215	30.239	33.263	36.287	42.335	48.383	76
77	24.510	27.574	30.637	33.701	36.765	42.892	49.020	77
78	24.828	27.932	31.035	34.139	37.242	43.449	49.656	78
79	25.146	28.290	31.433	34.576	37.720	44.006	50.293	79
80	25.465	28.648	31.831	35.014	38.197	44.563	50.930	80
81	25.783	29.006	32.229	35.452	38.675	45.120	51.566	81
82	26.101	29.364	32.627	35.889	39.152	45.677	52.203	82
83	26.420	29.722	33.025	36.327	39.630	46.235	52.839	83
84	26.738	30.080	33.423	36.765	40.107	46.792	53.476	84
85	27.056	30.438	33.820	37.202	40.585	47.349	54.113	85
86	27.375	30.796	34.218	37.640	41.062	47.906	54.749	86
87	27.693	31.155	34.616	38.078	41.539	48.463	55.386	87
88	28.011	31.513	35.014	38.515	42.017	49.020	56.023	88
89	28.330	31.871	35.412	38.953	42.494	49.577	56.659	89
90	28.648	32.229	35.810	39.391	42.972	50.134	57.296	90
91	28.966	32.587	36.208	39.829	43.449	50.691	57.932	91
92	29.285	32.945	36.606	40.266	43.927	51.248	58.569	92
93	29.603	33.303	37.004	40.704	44.404	51.805	59.206	93
94	29.921	33.661	37.401	41.142	44.882	52.362	59.842	94
95	30.239	34.019	37.799	41.579	45.359	52.919	60.479	95
96	30.558	34.377	38.197	42.017	45.837	53.476	61.116	96
97	30.876	34.736	38.595	42.455	46.314	54.033	61.752	97
98	31.194	35.094	38.993	42.892	46.792	54.590	62.389	98
99	31.513	35.452	39.391	43.330	47.269	55.147	63.025	99

Outside diameter of any gear is equal to the pitch diameter of a similar gear with two additional teeth.

Inside diameter of any gear is equal to the pitch diameter of a similar gear with two less teeth, minus the clearance.

These two rules do not hold for gears with less than 17 teeth.

TABLE 16e (*Continued*)

PITCH DIAMETERS OF GEARS FOR CIRCULAR PITCH.

No. of Teeth	Circular Pitch in Inches						No. of Teeth
	2¼	2½	2¾	3	3¼	3½	
12	8.594	9.549	10.504	11.459	12.414	13.369	12
13	9.311	10.345	11.380	12.414	13.449	14.483	13
14	10.027	11.141	12.255	13.369	14.483	15.597	14
15	10.743	11.937	13.130	14.324	15.518	16.711	15
16	11.459	12.732	14.006	15.279	16.552	17.825	16
17	12.175	13.528	14.881	16.234	17.587	18.939	17
18	12.892	14.324	15.756	17.189	18.621	20.054	18
19	13.608	15.120	16.632	18.144	19.656	21.168	19
20	14.324	15.915	17.507	19.099	20.690	22.282	20
21	15.040	16.711	18.382	20.054	21.725	23.396	21
22	15.756	17.507	19.258	21.008	22.759	24.510	22
23	16.473	18.303	20.133	21.963	23.794	25.624	23
24	17.189	19.099	21.008	22.918	24.828	26.738	24
25	17.905	19.894	21.884	23.873	25.863	27.852	25
26	18.621	20.690	22.759	24.828	26.897	28.966	26
27	19.337	21.486	23.635	25.783	27.932	30.080	27
28	20.054	22.282	24.510	26.738	28.966	31.194	28
29	20.770	23.077	25.385	27.693	30.001	32.308	29
30	21.486	23.873	26.261	28.648	31.035	33.423	30
31	22.202	24.669	27.136	29.603	32.070	34.537	31
32	22.918	25.465	28.011	30.558	33.104	35.651	32
33	23.635	26.261	28.887	31.513	34.139	36.765	33
34	24.351	27.056	29.762	32.468	35.173	37.879	34
35	25.067	27.852	30.637	33.423	36.208	38.993	35
36	25.783	28.648	31.513	34.377	37.242	40.107	36
37	26.499	29.444	32.388	35.332	38.277	41.221	37
38	27.215	30.239	33.263	36.287	39.311	42.335	38
39	27.932	31.035	34.139	37.242	40.346	43.449	39
40	28.648	31.831	35.014	38.197	41.380	44.563	40
41	29.364	32.627	35.889	39.152	42.415	45.677	41
42	30.080	33.423	36.765	40.107	43.449	46.792	42
43	30.796	34.218	37.640	41.062	44.484	47.906	43
44	31.513	35.014	38.515	42.017	45.518	49.020	44
45	32.229	35.810	39.391	42.972	46.553	50.134	45
46	32.945	36.606	40.266	43.927	47.587	51.248	46
47	33.661	37.401	41.142	44.882	48.622	52.362	47
48	34.377	38.197	42.017	45.837	49.656	53.476	48
49	35.094	38.993	42.892	46.792	50.691	54.590	49
50	35.810	39.789	43.768	47.746	51.725	55.704	50
51	36.526	40.585	44.643	48.701	52.760	56.818	51
52	37.242	41.380	45.518	49.656	53.794	57.932	52
53	37.958	42.176	46.394	50.611	54.829	59.046	53
54	38.675	42.972	47.269	51.566	55.863	60.161	54

Outside diameter of any gear is equal to the pitch diameter of a similar gear with two additional teeth.

Inside diameter of any gear is equal to the pitch diameter of a similar gear with two less teeth, minus the clearance.

These two rules do not hold for gears with less than 17 teeth.

TABLE 16e (*Continued*)

Pitch Diameters of Gears for Circular Pitch.

No. of Teeth	Circular Pitch in Inches						No. of Teeth
	2¼	2½	2¾	3	3¼	3½	
55	39.391	43.768	48.144	52.521	56.898	61.275	55
56	40.107	44.563	49.020	53.476	57.932	62.389	56
57	40.823	45.359	49.895	54.431	58.967	63.503	57
58	41.539	46.155	50.770	55.386	60.001	64.617	58
59	42.256	46.951	51.646	56.341	61.036	65.731	59
60	42.972	47.746	52.521	57.296	62.070	66.845	60
61	43.688	48.542	53.396	58.251	63.105	67.959	61
62	44.404	49.338	54.272	59.206	64.139	69.073	62
63	45.120	50.134	55.147	60.161	65.174	70.187	63
64	45.837	50.930	56.023	61.116	66.208	71.301	64
65	46.553	51.725	56.898	62.070	67.243	72.415	65
66	47.269	52.521	57.773	63.025	68.277	73.530	66
67	47.985	53.317	58.649	63.980	69.312	74.644	67
68	48.701	54.113	59.524	64.935	70.346	75.758	68
69	49.418	54.908	60.399	65.890	71.381	76.872	69
70	50.134	55.704	61.275	66.845	72.415	77.986	70
71	50.850	56.500	62.150	67.800	73.450	79.100	71
72	51.566	57.296	63.025	68.755	74.485	80.214	72
73	52.282	58.092	63.901	69.710	75.519	81.328	73
74	52.999	58.887	64.776	70.665	76.554	82.442	74
75	53.715	59.683	65.651	71.620	77.588	83.556	75
76	54.431	60.479	66.527	72.575	78.623	84.670	76
77	55.147	61.275	67.402	73.530	79.657	85.785	77
78	55.863	62.070	68.277	74.485	80.692	86.899	78
79	56.580	62.866	69.153	75.439	81.726	88.013	79
80	57.296	63.662	70.028	76.394	82.761	89.127	80
81	58.012	64.458	70.904	77.349	83.795	90.241	81
82	58.728	65.254	71.779	78.304	84.830	91.355	82
83	59.444	66.049	72.654	79.259	85.864	92.469	83
84	60.161	66.845	73.530	80.214	86.899	93.583	84
85	60.877	67.641	74.405	81.169	87.933	94.697	85
86	61.593	68.437	75.280	82.124	88.968	95.811	86
87	62.309	69.232	76.156	83.079	90.002	96.925	87
88	63.025	70.028	77.031	84.034	91.037	98.039	88
89	63.742	70.824	77.906	84.989	92.071	99.154	89
90	64.458	71.620	78.782	85.944	93.106	100.268	90
91	65.174	72.416	79.657	86.899	94.140	101.382	91
92	65.890	73.211	80.532	87.854	95.175	102.496	92
93	66.606	74.007	81.408	88.808	96.209	103.610	93
94	67.323	74.803	82.283	89.763	97.244	104.724	94
95	68.039	75.599	83.158	90.718	98.278	105.838	95
96	68.755	76.394	84.034	91.673	99.313	106.952	96
97	69.471	77.190	84.909	92.628	100.347	108.066	97
98	70.187	77.986	85.785	93.583	101.382	109.180	98
99	70.904	78.782	86.660	94.538	102.416	110.294	99

Outside diameter of any gear is equal to the pitch diameter of a similar gear with two additional teeth.

Inside diameter of any gear is equal to the pitch diameter of a similar gear with two less teeth, minus the clearance.

These two rules do not hold for gears with less than 17 teeth.

TABLE 16e (*Continued*)

PITCH DIAMETERS OF GEARS FOR CIRCULAR PITCH.

No. of Teeth	CIRCULAR PITCH IN INCHES						No. of Teeth
	3¾	4	4½	5	5½	6	
12	14.324	15.279	17.189	19.099	21.008	22.918	12
13	15.518	16.552	18.621	20.690	22.759	24.828	13
14	16.711	17.825	20.054	22.282	24.510	26.738	14
15	17.905	19.099	21.486	23.873	26.261	28.648	15
16	19.099	20.372	22.918	25.465	28.011	30.558	16
17	20.292	21.645	24.351	27.056	29.762	32.468	17
18	21.486	22.918	25.783	28.648	31.513	34.377	18
19	22.680	24.192	27.215	30.239	33.263	36.287	19
20	23.873	25.465	28.648	31.831	35.014	38.197	20
21	25.067	26.738	30.080	33.423	36.765	40.107	21
22	26.261	28.011	31.513	35.014	38.515	42.017	22
23	27.454	29.285	32.945	36.606	40.266	43.927	23
24	28.648	30.558	34.377	38.197	42.017	45.837	24
25	29.842	31.831	35.810	39.789	43.768	47.746	25
26	31.035	33.104	37.242	41.380	45.518	49.656	26
27	32.229	34.377	38.675	42.972	47.269	51.566	27
28	33.423	35.651	40.107	44.563	49.020	53.476	28
29	34.616	36.924	41.539	46.155	50.770	55.386	29
30	35.810	38.197	42.972	47.746	52.521	57.296	30
31	37.004	39.470	44.404	49.338	54.272	59.206	31
32	38.197	40.744	45.837	50.930	56.023	61.116	32
33	39.391	42.017	47.269	52.521	57.773	63.025	33
34	40.585	43.290	48.701	54.113	59.524	64.935	34
35	41.778	44.563	50.134	55.704	61.275	66.845	35
36	42.972	45.837	51.566	57.296	63.025	68.755	36
37	44.165	47.110	52.999	58.887	64.776	70.665	37
38	45.359	48.383	54.431	60.479	66.527	72.575	38
39	46.553	49.656	55.863	62.070	68.277	74.485	39
40	47.746	50.930	57.296	63.662	70.028	76.394	40
41	48.940	52.203	58.728	65.254	71.779	78.304	41
42	50.134	53.476	60.161	66.845	73.530	80.214	42
43	51.327	54.749	61.593	68.437	75.280	82.124	43
44	52.521	56.023	63.025	70.028	77.031	84.034	44
45	53.715	57.296	64.458	71.620	78.782	85.944	45
46	54.908	58.569	65.890	73.211	80.532	87.854	46
47	56.102	59.842	67.323	74.803	82.283	89.763	47
48	57.296	61.116	68.755	76.394	84.034	91.673	48
49	58.489	62.389	70.187	77.986	85.785	93.583	49
50	59.683	63.662	71.620	79.577	87.535	95.493	50
51	60.877	64.935	73.052	81.169	89.286	97.403	51
52	62.070	66.208	74.485	82.761	91.037	99.313	52
53	63.264	67.482	75.917	84.352	92.787	101.223	53
54	64.458	68.755	77.349	85.944	94.538	103.132	54

Outside diameter of any gear is equal to the pitch diameter of a similar gear with two additional teeth.

Inside diameter of any gear is equal to the pitch diameter of a similar gear with two less teeth, minus the clearance.

These two rules do not hold for gears with less than 17 teeth.

TABLE 16e (*Continued*)

PITCH DIAMETERS OF GEARS FOR CIRCULAR PITCH.

No. of Teeth	Circular Pitch in Inches						No. of Teeth
	3¾	4	4½	5	5½	6	
55	65.651	70.028	78.782	87.535	96.289	105.042	55
56	66.845	71.301	80.214	89.127	98.039	106.952	56
57	68.039	72.575	81.646	90.718	99.790	108.862	57
58	69.232	73.848	83.079	92.310	101.541	110.772	58
59	70.426	75.121	84.511	93.901	103.292	112.682	59
60	71.620	76.394	85.944	95.493	105.042	114.592	60
61	72.813	77.668	87.376	97.085	106.793	116.501	61
62	74.007	78.941	88.808	98.676	108.544	118.411	62
63	75.201	80.214	90.241	100.268	110.294	120.321	63
64	76.394	81.487	91.673	101.859	112.045	122.231	64
65	77.588	82.761	93.106	103.451	113.796	124.141	65
66	78.782	84.034	94.538	105.042	115.546	126.051	66
67	79.975	85.307	95.970	106.634	117.297	127.961	67
68	81.169	86.580	97.403	108.225	119.048	129.870	68
69	82.363	87.854	98.835	109.817	120.799	131.780	69
70	83.556	89.127	100.268	111.408	122.549	133.690	70
71	84.750	90.400	101.700	113.000	124.300	135.600	71
72	85.944	91.673	103.132	114.592	126.051	137.510	72
73	87.137	92.946	104.565	116.183	127.801	139.420	73
74	88.331	94.220	105.997	117.775	129.552	141.330	74
75	89.525	95.943	107.430	119.366	131.303	143.239	75
76	90.718	96.766	108.862	120.958	133.054	145.149	76
77	91.912	98.039	110.294	122.549	134.804	147.059	77
78	93.106	99.313	111.727	124.141	136.555	148.969	78
79	94.299	100.586	113.159	125.732	138.306	150.879	79
80	95.493	101.859	114.592	127.324	140.056	152.789	80
81	96.687	103.132	116.024	128.916	141.807	154.699	81
82	97.880	104.406	117.456	130.507	143.558	156.608	82
83	99.074	105.679	118.889	132.099	145.308	158.518	83
84	100.268	106.952	120.321	133.690	147.059	160.428	84
85	101.461	108.225	121.754	135.282	148.810	162.338	85
86	102.655	109.499	123.186	136.873	150.561	164.248	86
87	103.849	110.772	124.618	138.465	152.311	166.158	87
88	105.042	112.045	126.051	140.056	154.062	168.068	88
89	106.236	113.318	127.483	141.648	155.813	169.977	89
90	107.430	114.592	128.916	143.239	157.563	171.887	90
91	108.623	115.865	130.348	144.831	159.314	173.797	91
92	109.817	117.138	131.780	146.423	161.065	175.707	92
93	111.011	118.411	133.213	148.014	162.816	177.617	93
94	112.204	119.685	134.645	149.606	164.566	179.527	94
95	113.398	120.958	136.077	151.197	166.317	181.437	95
96	114.592	122.231	137.510	152.789	168.068	183.347	96
97	115.785	123.504	138.942	154.380	169.818	185.256	97
98	116.979	124.777	140.375	155.972	171.569	187.167	98
99	118.173	126.051	141.807	157.563	173.320	189.076	99

Outside diameter of any gear is equal to the pitch diameter of a similar gear with two additional teeth.

Inside diameter of any gear is equal to the pitch diameter of a similar gear with two less teeth, minus the clearance.

These two rules do not hold for gears with less than 17 teeth.

TABLE 16*f*

INTENSITIES FOR COMPRESSION MEMBERS OF CARBON STEEL WITH FIXED ENDS.

$$P = 16{,}000 - 60\frac{l}{r}.$$

$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P
1	15940	26	14440	51	12940	76	11440	101	9940	126	8440
2	15880	27	14380	52	12880	77	11380	102	9880	127	8380
3	15820	28	14320	53	12820	78	11320	103	9820	128	8320
4	15760	29	14260	54	12760	79	11260	104	9760	129	8260
5	15700	30	14200	55	12700	80	11200	105	9700	130	8200
6	15640	31	14140	56	12640	71	11140	106	9640	131	8140
7	15580	32	14080	57	12580	82	11080	107	9580	132	8080
8	15520	33	14020	58	12520	83	11020	108	9520	133	8020
9	15460	34	13960	59	12460	84	10960	109	9460	134	7960
10	15400	35	13900	60	12400	85	10900	110	9400	135	7900
11	15340	36	13840	61	12340	86	10840	111	9340	136	7840
12	15280	37	13780	62	12280	87	10780	112	9280	137	7780
13	15220	38	13720	63	12220	88	10720	113	9220	138	7720
14	15160	39	13660	64	12160	89	10660	114	9160	139	7660
15	15100	40	13600	65	12100	90	10600	115	9100	140	7600
16	15040	41	13540	66	12040	91	10540	116	9040	141	7540
17	14980	42	13480	67	11980	92	10480	117	8980	142	7480
18	14920	43	13420	68	11920	93	10420	118	8920	143	7420
19	14860	44	13360	69	11860	94	10360	119	8860	144	7360
20	14800	45	13300	70	11800	95	10300	120	8800	145	7300
21	14740	46	13240	71	11740	96	10240	121	8740	146	7240
22	14680	47	13180	72	11680	97	10180	122	8680	147	7180
23	14620	48	13120	73	11620	98	10120	123	8620	148	7120
24	14560	49	13060	74	11560	99	10060	124	8560	149	7060
25	14500	50	13000	75	11500	100	10000	125	8500	150	7000

TABLE 16*g*

INTENSITIES FOR COMPRESSION MEMBERS OF CARBON STEEL WITH HINGED ENDS.

$$P = 16{,}000 - 80\frac{l}{r}.$$

$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P
1	15920	25	14000	49	12080	73	10160	97	8240
2	15840	26	13920	50	12000	74	10080	98	8160
3	15760	27	13840	51	11920	75	10000	99	8080
4	15680	28	13760	52	11840	76	9920	100	8000
5	15600	29	13680	53	11760	77	9840	101	7920
6	15520	30	13600	54	11680	78	9760	102	7840
7	15440	31	13520	55	11600	79	9680	103	7760
8	15360	32	13440	56	11520	80	9600	104	7680
9	15280	33	13360	57	11440	81	9520	105	7600
10	15200	34	13280	58	11360	82	9440	106	7520
11	15120	35	13200	59	11280	83	9360	107	7440
12	15040	36	13120	60	11200	84	9280	108	7360
13	14930	37	13040	61	11120	85	9200	109	7280
14	14880	38	12960	62	11040	86	9120	110	7200
15	14800	39	12880	63	10960	87	9040	111	7120
16	14720	40	12800	64	10880	88	8960	112	7040
17	14640	41	12720	65	10800	89	8880	113	6960
18	14560	42	12640	66	10720	90	8800	114	6880
19	14480	43	12560	67	10640	91	8720	115	6800
20	14400	44	12480	68	10560	92	8640	116	6720
21	14320	45	12400	69	10480	93	8560	117	6640
22	14240	46	12320	70	10400	94	8480	118	6560
23	14160	47	12240	71	10320	95	8400	119	6480
24	14080	48	12160	72	10240	96	8320	120	6400

TABLE 16*h*

INTENSITIES FOR FORKED ENDS AND EXTENSION-PLATES OF COMPRESSION MEMBERS OF CARBON STEEL.

Formula: $P = 10{,}000 - 300\frac{l}{t}$.

$\frac{l}{t}$	P	$\frac{l}{t}$	P	$\frac{l}{t}$	P
1	9700	11	6700	21	3700
2	9400	12	6400	22	3400
3	9100	13	6100	23	3100
4	8800	14	5800	24	2800
5	8500	15	5500	25	2500
6	8200	16	5200	26	2200
7	7900	17	4900	27	1900
8	7600	18	4600	28	1600
9	7300	19	4300	29	1300
10	7000	20	4000	30	1000

TABLE 16*j*

BEARING VALUES FOR CARBON STEEL PINS.

Diam. of Pin. (Ins.)	BEARING		Diam. of Pin (Ins.)	BEARING		Diam. of Pin (Ins.)	BEARING	
	22,000 Lbs. per Sq. In.	28,600 Lbs. per Sq. In.		22,000 Lbs. per Sq. In.	28,600 Lbs. per Sq. In.		22,000 Lbs. per Sq. In.	28,600 Lbs. per Sq. In.
2	44,000	57,200	7¼	160,000	207,000	12½	275,000	357,000
2¼	49,500	64,400	7½	165,000	215,000	12¾	281,000	365,000
2½	55,000	71,500	7¾	171,000	222,000	13	286,000	372,000
2¾	60,500	78,700	8	176,000	229,000	13¼	292,000	379,000
3	66,000	85,800	8¼	182,000	236,000	13½	297,000	386,000
3¼	71,500	93,000	8½	187,000	243,000	13¾	303,000	393,000
3½	77,000	100,100	8¾	193,000	250,000	14	308,000	400,000
3¾	82,500	107,300	9	198,000	257,000	14¼	314,000	408,000
4	88,000	114,400	9¼	204,000	265,000	14½	319,000	415,000
4¼	93,500	121,600	9½	209,000	272,000	14¾	325,000	422,000
4½	99,000	128,700	9¾	215,000	279,000	15	330,000	429,000
4¾	104,500	135,900	10	220,000	286,000	15½	341,000	443,000
5	110,000	143,000	10¼	226,000	293,000	16	352,000	458,000
5¼	116,000	150,000	10½	231,000	300,000	16½	363,000	472,000
5½	121,000	157,000	10¾	237,000	308,000	17	374,000	486,000
5¾	127,000	165,000	11	242,000	315,000	17½	385,000	500,000
6	132,000	172,000	11¼	248,000	322,000	18	396,000	515,000
6¼	138,000	179,000	11½	253,000	329,000	18½	407,000	529,000
6½	143,000	186,000	11¾	259,000	336,000	19	418,000	543,000
6¾	149,000	193,000	12	264,000	343,000	19½	429,000	558,000
7	154,000	200,000	12¼	270,000	350,000	20	440,000	572,000

NOTE.—22,000 lbs. per sq. in. is the allowable stress excluding wind.
28,600 lbs. per sq. in. is the allowable stress including wind.

TABLE 16k

ALLOWABLE BENDING MOMENTS ON PINS OF CARBON STEEL AND NICKEL STEEL.

Diam. of Pin (Ins.)	Moments in Inch Pounds for Fibre Stress of			Diam. of Pin (Inches)	Moments in Inch Pounds for Fibre Stress of			Diam. of Pin (Inches)	Moments in Inch Pounds for Fibre Stress of		
	27,000 Lbs. per Sq. In.	35,100 Lbs. per Sq. In.	45,000 Lbs. per Sq. In.		27,000 Lbs. per Sq. In.	35,100 Lbs. per Sq. In.	45,000 Lbs. per Sq. In.		27,000 Lbs. per Sq. In.	35,100 Lbs. per Sq. In.	45,000 Lbs. per Sq. In.
2	21,200	27,600	35,300	7¼	1,010,000	1,313,000	1,683,000	12½	5,177,000	6,730,000	8,628,000
2¼	30,200	39,300	50,300	7½	1,118,000	1,454,000	1,864,000	12¾	5,494,000	7,142,000	9,157,000
2½	41,400	53,800	69,000	7¾	1,234,000	1,604,000	2,057,000	13	5,824,000	7,571,000	9,707,000
2¾	55,100	71,700	91,800	8	1,357,000	1,764,000	2,262,000	13¼	6,166,000	8,016,000	10,277,000
3	71,600	93,100	119,300	8¼	1,489,000	1,935,000	2,481,000	13½	6,522,000	8,478,000	10,870,000
3¼	91,000	118,300	151,700	8½	1,628,000	2,116,000	2,713,000	13¾	6,891,000	8,958,000	11,485,000
3½	113,600	147,700	189,300	8¾	1,776,000	2,308,000	2,960,000	14	7,274,000	9,456,000	12,123,000
3¾	139,800	181,700	233,000	9	1,932,000	2,512,000	3,220,000	14¼	7,671,000	9,972,000	12,785,000
4	170,000	221,000	283,000	9¼	2,098,000	2,727,000	3,497,000	14½	8,081,000	10,505,000	13,468,000
4¼	204,000	265,000	340,000	9½	2,273,000	2,955,000	3,788,000	14¾	8,506,000	11,058,000	14,177,000
4½	242,000	314,000	403,000	9¾	2,457,000	3,194,000	4,095,000	15	8,946,000	11,630,000	14,910,000
4¾	284,000	369,000	473,000	10	2,651,000	3,446,000	4,418,000	15½	9,871,000	12,833,000	16,452,000
5	331,000	431,000	552,000	10¼	2,854,000	3,711,000	4,757,000	16	10,857,000	14,114,000	18,095,000
5¼	384,000	499,000	640,000	10½	3,069,000	3,989,000	5,114,000	16½	11,907,000	15,480,000	19,845,000
5½	441,000	573,000	735,000	10¾	3,293,000	4,281,000	5,488,000	17	13,023,000	16,930,000	21,705,000
5¾	504,000	655,000	840,000	11	3,528,000	4,587,000	5,880,000	17½	14,206,000	18,468,000	23,677,000
6	573,000	744,000	954,000	11¼	3,774,000	4,907,000	6,290,000	18	15,459,000	20,097,000	25,765,000
6¼	647,000	841,000	1,078,000	11½	4,031,000	5,241,000	6,719,000	18½	16,784,000	21,819,000	27,973,000
6½	728,000	946,000	1,213,000	11¾	4,300,000	5,590,000	7,167,000	19	18,181,000	23,636,000	30,302,000
6¾	815,000	1,060,000	1,359,000	12	4,581,000	5,955,000	7,635,000	19½	19,655,000	25,551,000	32,758,000
7	909,000	1,182,000	1,515,000	12¼	4,873,000	6,335,000	8,122,000	20	21,206,000	27,568,000	35,343,000

NOTE.—27,000 lbs. per sq. in. is the allowable stress for carbon steel excluding wind.
35,100 lbs. per sq. in. is the allowable stress for carbon steel including wind.
45,000 lbs. per sq. in. is the allowable stress for nickel steel excluding wind.

TABLE 16i

SHEARING AND BEARING VALUES FOR CARBON STEEL RIVETS.

Diam. of Rivet (Inches)	Single Shear 10,000 S 8,000 F (Pounds)	Unit Bearing Value (Pounds)	Shop or Field	BEARING VALUES FOR DIFFERENT THICKNESSES OF PLATES (Pounds)												
				¼ In.	5/16 In.	⅜ In.	7/16 In.	½ In.	9/16 In.	⅝ In.	11/16 In.	¾ In.	13/16 In.	⅞ In.	15/16 In.	1 In.
½ . .	S : 1,963	20,000	S	2,500	3,130	3,750	4,380	.....	.....	.....	.....	.....	.....	.....	.....	.....
	F : 1,570	16,000	F	2,000	2,500	3,000	3,500	.....	.....	.....	.....	.....	.....	.....	.....	.....
⅝ . .	S : 3,068	20,000	S	3,130	3,910	4,690	5,470	6,250	7,030	.....	.....	.....	.....	.....	.....	.....
	F : 2,454	16,000	F	2,500	3,130	3,750	4,380	5,000	5,630	.....	.....	.....	.....	.....	.....	.....
¾ . .	S : 4,418	20,000	S	3,750	4,690	5,630	6,560	7,500	8,440	9,380	10,310	.....	.....	.....	.....	.....
	F : 3,534	16,000	F	3,000	3,750	4,500	5,250	6,000	6,750	7,500	8,250	.....	.....	.....	.....	.....
⅞ . .	S : 6,013	20,000	S	4,380	5,470	6,560	7,660	8,750	9,840	10,940	12,030	13,130	14,220	.....	.....	.....
	F : 4,810	16,000	F	3,500	4,380	5,250	6,130	7,000	7,880	8,750	9,630	10,500	11,380	.....	.....	.....
1 . .	S : 7,854	20,000	S	5,000	6,250	7,500	8,750	10,000	11,250	12,500	13,750	15,000	16,250	17,500	18,750	.....
	F : 6,283	16,000	F	4,000	5,000	6,000	7,000	8,000	9,000	10,000	11,000	12,000	13,000	14,000	15,000	.....
1⅛ . .	S : 9,940	20,000	S	5,630	7,030	8,440	9,840	11,250	12,660	14,060	15,470	16,880	18,280	19,690	21,090	22,500
	F : 7,952	16,000	F	4,500	5,630	6,750	7,880	9,000	10,130	11,250	12,380	13,500	14,630	15,750	16,875	18,000
1¼ . .	S : 12,272	20,000	S	6,250	7,810	9,380	10,940	12,500	14,060	15,630	17,190	18,750	20,310	21,870	23,440	25,000
	F : 9,818	16,000	F	5,000	6,250	7,500	8,750	10,000	11,250	12,500	13,750	15,000	16,250	17,500	18,750	20,000

S = Shop-driven rivets. F = Field-driven rivets.

CHAPTER XVII

SHOPWORK AS AFFECTING BRIDGE DESIGN

Ever since metal bridgework was started, there has been a conflict of opinion between the engineers and draftsmen of the independent or consulting bridge engineers and those of the manufacturers of structural metalwork. The consulting bridge engineers and their assistants look at everything from the point of view of obtaining results, perhaps without full appreciation of the economics involved; while the manufacturers and their assistants, on the other hand, are constantly bearing in mind the necessity of keeping down the cost of manufacture and erection in every possible way so as to make the tonnage of output as great and the time of erection as small as practicable, often irrespective of the true interests of the purchaser. Those on one side of the question say: "We want the work to be right no matter what it may cost," and those on the other side remark: "What is the sense in paying for useless frills and furbelows? The regular, general product of our shops ought to be good enough for anybody." Of late years the consulting engineers and the engineers of the manufacturing companies have gradually been getting into better accord, as have also their assistants, because many of them have had experience in both lines of work.

That the agreement is far better to-day than it was two or three decades ago nobody can deny; for in the old days it was almost obligatory to leave most of the detailing to the shops. The author recalls distinctly the first set of plans for important bridgework that he ever prepared and the trouble in which they involved him. They were for three high, wrought-iron towers, each having four battered faces. The complete set of plans was designed, drafted, and traced by him with his own hands. Being entirely ignorant of everything connected with shopwork, he adopted the academic method of development for showing the tower legs so as to illustrate clearly (at least to himself) the oblique cuts at top and bottom for the attachment of the bearing plates. The entire set of plans was returned by the shops with the remark that nobody there, from their chief engineer down, could understand that kind of drafting or had ever seen or even heard of it before; and it was necessary to make a new set in which only ordinary projection was employed, laying out every line of each column and brace and marking thereon its length calculated to the nearest sixty-fourth of an inch so as to determine the oblique cuts at the ends.

But in spite of all improvements in the conditions, the consulting and the manufacturing engineers have not yet quite gotten together on all important points. Recognizing this, the author some months ago wrote to several of the leading engineers engaged in structural steel manufacture asking them to state for publication in this treatise the principal points wherein they differ in opinion with the consulting bridge engineers in general, the intention being to bring all parties into still closer accord. Only two of the gentlemen communicated with were able to comply with the request; but, fortunately, they are two of the highest authorities in their line in America, viz., Paul Wolfel, Esq., C.E., Chief Engineer of the McClintic-Marshall Company, and Albert F. Reichmann, Esq., C.E., Division Engineer of the American Bridge Company. Their communications will be reproduced practically verbatim; and then the author will take the liberty of stating those points on which he, as a consulting engineer, still has to differ with them after giving due and thoughtful consideration to all that they have said. He desires here to express to them his great appreciation of their kindness in complying with his request, undoubtedly at the expense of much of their valuable time and energy, and also to acknowledge the great worth of their opinions on all practical matters connected with the manufacture of structural steel.

Mr. Wolfel sent the following statement:

"POINTS TO BE CONSIDERED IN ORDER TO GET THE BEST RESULTS WITH THE LEAST EXPENSE IN DESIGNING STEEL STRUCTURES.

"(A.) *Material.*

"(1.) Bear in mind that all angles over 6″ and all beams over 15″ deep cost $2.00 per ton above the base price.

"(2.) Bear in mind that for large plates the following extras govern:

Plates	over	100″	up to	110″,	inclusive,	$1.00	per	ton
"	"	110″	" "	115″	"	2.00	"	"
"	"	115″	" "	120″	"	3.00	"	"
"	"	120″	" "	125″	"	5.00	"	"
"	"	125″	" "	130″	"	10.00	"	"
"	"	130″	" "	....	...	20.00	"	"

"Therefore if you design a plate girder where a 124″ web will undoubtedly answer just as well, do not use a 126″ web, as this will cost you $5.00 per ton of material more than you would have to pay for the 124″ web.

"(B.) *Shopwork.*

"(1.) Make your design so that all extra operations are avoided. This particularly should apply to large and heavy members or to small members used in large numbers. The work on these pieces should be kept as simple as possible, and any odd work that has to be done should be confined to small pieces not required in large numbers.

"(2.) Do not make the work subpunched and reamed unless necessary. Particularly, do not subpunch and ream laterals, lateral connections, tie-plates, lattice bars, diaphragms, and other minor details.

"(3.) Do not call for work drilled from the solid unless absolutely necessary or unless by so doing savings otherwise can be obtained.

"(4.) Reaming to templets is useless unless the templets can be set from finished surfaces, as in chord splices, ends of stringers, floor-beam connections, etc. Reaming of laterals to templets is liable to do more harm than good, as no finished surfaces are available. The same applies to diagonals in trusswork. While in a punched connection a few holes may be slightly out, which can be corrected in the field, if a connection is reamed to templet and the templet is not properly set, all holes will be equally out. Riveted trusses should be reamed in the maker's shop assembled, and when assembled match-marked.

"(5.) Never call for edges of plates to be planed where a sheared edge will answer, or where the thicknesses are such that shearing can be neatly done.

"(6.) Plate girders with cover plates up to 16″ in width, inclusive, should always be designed with two lines of rivets in these cover plates. This greatly helps in the shop and materially simplifies the notching of the ties.

"(7.) In all plate girders and truss stringers the lateral system should be sufficiently dropped so that the rivet heads of the same clear the ties.

"(8.) Wherever possible in heavy work, avoid, in the construction of the chords or web members, side plates or doubling up of the web plates. It will often pay to use heavier web plates without side plates, even if they have to be drilled from the solid. If, however, webs have to be doubled up or side plates used, the stitch rivets made necessary by this construction should be reduced to a reasonable amount. If a plate is used as a cover plate in a chord, it is good practice to limit its thickness to 1/40th of the distance between rivets. If the same plate was used as a side plate in a chord, in most designs two or three times as many lines of rivets would be called for than would be necessary by the above limits.

"(9.) It is cheaper and better to use heavy chord angles in stringers than lighter chord angles with cover plates. Even if the chord angles, on account of the increased thickness, should have to be drilled from the solid, this will hold good.

"(10.) Avoid all beveled cuts wherever possible, especially beveled cuts for angles that cannot be obtained by cutting multiple pieces from a long piece; also beveled cuts in all beams and channels, as these have to be sawed.

"(11.) One of the biggest savings in recent years in bridge shops has been made by the use of multiple punches. These not only reduce the cost of the punching proper, but also save the cost of making templets and the laying out of the material. They further give far superior work; as the effect of the stretch of the material during punching on the accuracy of the work is eliminated, if these multiple punches are properly constructed. Their use, therefore, should be encouraged in every way. In order to do this, it is necessary to:

"(*a.*) Keep all rivets in line longitudinally.

"(*b.*) Keep as many rivets in line transversely as possible and do not use any more combinations of rivets transversely than necessary.

"(*c.*) Never have the longitudinal lines of rivets less than 2¼″ apart, nor the transverse lines less than 1¼″.

"(12.) Do not crimp stiffeners if it can be helped, especially do not crimp stiffeners of short lengths, say up to about three feet. If stiffeners are crimped ¾″ or more, the crimp is unsightly; and better and more sightly work will be obtained by using a thin filler with a smaller crimp. Do not call for fillers or splice plates to have a tight fit, as this is impracticable in the shop. The stiffeners, of course, should have a close bearing. Do not attempt to use stiffeners on rolled beams, as it is practically impossible to give them a tight fit against the flanges.

"(13.) Do not call for planing of the base, cap, sole, or masonry plates, as the mills can roll the same closer than they can be planed.

"(14.) Abolish the old rule of 60 degs. for single lacing, as this makes the lacing entirely too close for narrow members and is quite expensive and unsightly. In built members of a box section, formed by either rolled or built-up channels, wherever possible turn the flanges out. This applies particularly to deck spans, where nothing whatever is gained by turning the flanges in.

"(15.) Avoid all hand riveting wherever possible, also all odd riveting that has to be done either before the work is assembled or after a piece is finished otherwise.

"(C.) *Design.*

"(1.) On short plate-girder spans omit the bottom lateral system.

"(2.) On plate girders of medium length, vary the size of the lateral angles as little as possible, taking care of the variation of the section by varying the thickness.

"(3.) Do not use lug angles on the laterals if it can be avoided.

"(4.) In most cases it will simplify the shopwork not to run the lateral angles in on the chord angles of the girder.

"(5.) Avoid the use of the side plates wherever possible. If they have to be used, arrange them as per sketch (1) or (2) but never as per sketch (3). (See Fig. 17*a*.)

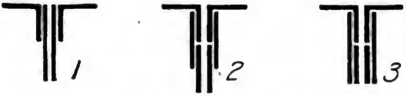


Fig. 17*a*. Types of Built Flanges for Plate-girder Spans.

"(6.) For viaducts keep the girder spans over the towers and between the towers of the same depth, as a shallow girder over the tower greatly complicates the erection.

"(7.) In laying out viaducts, keep as many towers alike as possible by varying your foundations in height as far as practicable.

"(8.) In square girders use an even number of panels for the bracing; in skew girders an odd number.

"(9.) The greatest amount of duplication in skew spans will be obtained if the floor is laid out so that the entire span can be revolved around a central point.

"(10.) In riveted truss spans the simplest construction will be obtained if the end pin is kept below the bottom chord.

"(11.) If end floor-beams are desired, a simpler and better construction is obtained by supporting the end floor-beams directly on shoes and rollers and not riveting them into the truss, unless the same has vertical end posts.

"(12.) In pin connected work, do not vary the size of pins more than necessary.

"(13.) Where stringers frame into the floor-beams, if possible arrange it so that the stringer connection does not run over the flange angles of the floor-beams.

"(14.) In riveted tension members use tie-plates in place of lacing.

"(15.) In deck trusses and double-track viaducts, support the stringers and longitudinal girders on top of the floor-beams and don't frame them in. This avoids all complication as to expansion in viaducts.

"(16.) Do not call for a bearing of the web plates of girders on the sole plate.

"(17.) In deep girders do not make the web thickness less than 1/160 of the depth of the girder, as otherwise the webs are liable to buckle.

"(18.) Where webs are spliced, allow ample clearance and keep the depth of the web ½" less than the distance out to out of flange angles.

"(19.) Unless milled joints are called for in the chords or web members of trusses, allow clearance where the webs are spliced to the gussets or to themselves.

"(20.) Round corners in plate girders are expensive. Cambering of plate girders, especially short girders, is useless and quite expensive.

"(D.) *Field Work.*

"(1.) Allow ample clearance for all entering connections.

"(2.) Provide erection shelves for girders and beams, particularly when they frame opposite each other.

"(3.) Have all riveting arranged in such a way that it can follow the erection of the work. Riveting should never be allowed to interfere with the raising and placing of the steel.

"(4.) Be sure that cross frames in deck bridges can be swung in place without spreading the girders.

"(5.) Be careful in through plate girders to arrange your stiffeners so that the floor-beams and stringers can be put in place with the girders in their final position.

"(6.) Arrange your riveting around the ends of the spans so that the rivets can be driven with the steel in the final position. It should never be necessary to jack up spans to drive rivets.

"(7.) In trough floorwork, especially where the under clearance is small, arrange your design so that all field rivets can be driven from the top, as in Fig. 17*b*.

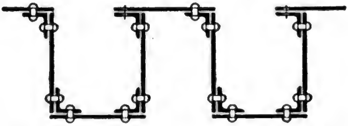


Fig. 17*b*. Trough Floor Construction for Easy Field Riveting.

"(8.) While it has been customary to call for 25 per cent excess for all field rivets 10 per cent excess should be sufficient if the rivets are driven with air hammers.

"(E.) *General.*

"(1.) In preparing the stress sheets, plus should be used for tension and minus for compression, as in figuring deflections the plus stress will then be applied to the member the length of which will be increased by the elastic deformation.

"(2.) Constant unit stresses should be used for tension, compression, shear, bearing, etc., and the stresses varied by adding impact. This greatly simplifies the detailing and makes it more rational, as it results in taking care of a certain number of square inches in tension always by the same number of square inches in shear and in bearing, etc."

Mr. Reichmann's communication reads as follows:

"SHOPWORK AS AFFECTING BRIDGE DESIGN.

"Attention is called to various details which will enable the shop to handle material to better advantage, reduce labor and fabrication, and generally expedite the work. All of these items are very important in lowering the fabrication costs and in giving quicker deliveries. Some of these details, because of their simplicity, also permit of better shopwork. For instance, when power-driven rivets can be used instead of hand-driven rivets, the finished member is more efficient; and members that consist of a small number of pieces are better than those containing a greater number.

"It is very important in substitutions of material as given herewith that the strength, economy, and efficiency of the member be not impaired by such substitutions. Other elements enter into this discussion, such as greater facilities of erection, cheaper maintenance, accessibility for repairs, etc. The various notes given are subdivided under different headings for easy reference. All points which apply to different classes of structures are placed under General and others are divided under the headings of Structural Bridgework, Bridge Machinery, and Viaducts. Types of work not covered by these classifications are usually of such special detail and design that it will not be necessary to note them except in a general way.

"GENERAL.

"Instead of using close lacing on compression members, it is often advisable to employ a solid web. This will sometimes permit the use of a lighter weight of main angles by counting the web as part of the section, and besides it will greatly facilitate the painting.

"For tension members use tie-plates instead of lacing bars. The exception to this is for members which require the lacing to prevent sagging, and the end bottom chords of trusses. The tie-plates have the advantage of getting better shop rivets; as, when the latter pass through lacing bars, they are difficult to drive.

"When designing lateral angles of small section, omit the lug angles at connections. For the lighter angles it is not important to develop both legs in tension at the connections. Generally, the ends of laterals, diagonals, etc., should be designed square, so that the mill lengths can be used without additional shearing and trimming. Channel diagonals of truss spans when turned out and exposed to view should, however, be cut on a bevel. When turned in and not exposed, the ends should be square, if possible.

"For box-shaped columns, diagonals, chords, etc., turn the angles or channels out, where permissible, to facilitate driving of rivets. In this connection it may be stated that the rivets can then be driven by power instead of by hand, which would not be the case if the angles or channels were turned in.

"For lacing, the lacing bars should be lapped generally, as this detail will save about half of the rivets. A common error is to design heavy compression members with the lacing bars having only one rivet at the ends. A better design would be to use wide lacing bars with two rivets at the end, and have the lacing in the heavier members reinforced with diaphragms placed a proper distance apart.

"Very often it is not necessary to mill the ends of I-beams, and all requirements are fulfilled by setting the I-beams back from the face of the connection angles. The designer should remember that the bridge shops are prepared to rivet on connection angles and get them square with a variation in the length of the beam not greater than 1/32" where milling is waived. The object of milling is to secure the right length and square ends, but this object can be obtained without the use of milling. Connection angles should never be fitted between the flanges of the I-beams, as the beams of the same section vary in size, and as it causes extra work to grind the connection angles to fit. Sometimes angles are fitted between flanges of I-beams to carry concentrated loads. This detail, because of its uncertainty in action, should be avoided, where the design will permit, by using heavy beams or more beams to distribute the loads.

"Fitted stiffeners, where not required for carrying a bearing, should be avoided. The stiffeners fitted at one end are particularly difficult to fit up so as to secure good results in the shop.

"Never bend plates on the width. When it is necessary to use plates which are circular in elevation cut the pieces out of plates of large enough dimensions so that the bending is not necessary. It is clear in this case that the bending of plates on the width is very difficult to attain because of the stretching on the tension side and the buckling on the compression side.

"The designer should always remember to allow plenty of clearance at the ends of sheared members so as to take up variations of shopwork. One-half inch clearance is considered a minimum for sheared ends. For entering connections, plenty of clearance should be allowed, as a great many of the difficulties in erection are due to lack of clearance; besides, giving reasonable clearance will permit of more rapid shopwork and the avoidance of many errors in the field due to inaccuracies thereof.

"Good results are obtained by straightening base, sole, and cap-plates—instead of planing them—up to $1\frac{1}{2}''$ thick. Beyond this thickness the plates should be planed.

"Avoid crimping long flange angles, as it is very difficult to handle a piece of great length in the crimping press.

"On floor-plates, skew-back angles, stiffening channels, etc., space the rivets 8 to 12". On some classes of work the spacing of rivets may be even greater, all depending upon the conditions under which the material is used. Eliminate excessive numbers of rivets. Study each line of rivets and reduce the number of those which are not required to transmit stress.

"For a great many purposes an angle railing will be satisfactory. In such cases avoid the use of the more expensive gas-pipe railing.

"When designing the size of stiffener angles to transmit bearing, as at the end of a plate-girder, the outstanding leg only should be counted for bearing, because of the variations in the fillets and the uncertainty of getting a tight fit.

"For tension bracing, in designing, give preference to angle bracing instead of rod bracing. It is possible thereby to use simpler connections, and the structure is also more rigid. Care, however, should be taken that the vertical leg is large enough so that the sagging due to its own weight is not excessive. In detailing the bracing a draw is allowed in the lengths of the pieces so that the angles may be pulled up taut.

"While this article does not discuss the subject of expansion, it is often advisable to provide, in addition to the usual allowance for expansion, a small amount for erecting the metal, due to what is called the 'growth of steel.' For viaducts such adjustment should be provided about every 400 feet; and similarly for mill-building work it is necessary to consider effects of adjustment, even if it is decided not to take care of expansion, while in other structures the joints for expansion will also serve the purpose of adjustment.

"Designs should be made to afford the bridge shops every facility possible for using their machinery to advantage. For instance, details should be arranged for multiple punch spacing, and to suit the requirements for bending, machining, and the various other operations which are governed by the shop equipment.

"The expansion points for stringers, or elevated railroad-girders where pockets are used, sometimes have not enough space behind the end stiffeners of the expansion girders to allow for the insertion of rivets through the end connections of the fixed girders. It should be remembered that the expansion and the fixed-end stringers are erected before the rivets in the connection of the fixed stringers are driven.

"When columns are set to stone bolts, which have been imbedded in masonry, the holes should be $\frac{3}{4}''$ or 1" larger than the diameter of the bolt so as to provide adjustment to take care of the inaccuracies in setting the bolts in the concrete.

"In the design of structural work for all purposes, more consideration should be given by the designer to the sections which are used. Special material should be avoided, if possible, sections varying by $\frac{1}{16}$ inch should be so combined as to use one section as far as possible, and special sections of small quantities should be eliminated entirely. Very often the delivery on the contract is delayed because the shop has to wait for a small quantity of a special section which is not rolled on time. Compliance with the above will insure better deliveries from the mill and quicker fabrication in the shop; and all parties concerned will be benefited thereby.

"When ordering plates, the designer should adhere to standard dimensions as far as possible. This can always be done in the case of lateral- and gusset-plates, but a special width may be necessary at times for the webs of stringer or girders. Standard widths for plates are 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 18, 20, 24, 30, 36, and 48 inches.

"Plates with beveled cuts should be designed with the idea of cutting in the shop in the most economical manner with the least number of cuts and the least waste.

"In determining what girder spans should be cambered, we would recommend that camber be omitted on all plate-girders under 75 ft. in length.

"For the camber of trusses up to 200 ft. increase the top chord in length ⅛" per 10 ft. For trusses over 200 ft. we would recommend figuring the actual deformations for each member and correcting the lengths of the members in accordance with same.

"To secure a substantial and practical pipe handrailing, the pipes for the posts should be threaded for fittings, and the rails should be fastened to the fittings with pins. The rails are ordered in random lengths and are held together between the posts with couplings.

"Standards for the gauges of angles will not be adaptable for all cases. To enable the shop to use a gauge-setting machine, gauges of 1⅛", 1⅜", 1¾", 2", 2¼", 2½", 3", 3½", and 4¾" should be used wherever possible.

"Information should be furnished to the fabricating shop, specifying the end of the structure which is to be erected first, it being very desirable to fabricate the work in the order of erection and also to note the direction of long plate-girders, so as to save turning them during shipment, at the shop, or in the field.

"For members with material from ¾" to 1" thick, or where the grips of rivets exceed 4", use 1" instead of ⅞" rivets. As the 1" rivets are about 30 per cent better than the ⅞" rivets, their number should be decreased in about the same proportion.

"Structural Bridgework.

"On chord or column sections extending over two panels with the same depth of section, but with smaller area required, the increased weight of the shop splices will offset the increase in weight due to making both sections the same, the big advantage in the latter, of course, being that the material is continuous without the splice.

"Frequently, on stringers and light girders, the webs are designed very light, which necessitates the use of many stiffeners to prevent the buckling of the webs. It is often a big advantage to thicken the web and omit the stiffeners. The weight in either case is about the same, as the omission of the stiffeners will offset the increased weight of the thicker web.

"For chord sections, the employment of reinforcing plates between angles should be avoided by using additional web-plates the full depth of the chord. This design has the advantage of connecting more of the main material to the flange angles direct, and avoids the use of a great many rivets which are necessary to connect the reinforcing plates to webs. When two webs are riveted together the rivets should be about 12" from centre to centre, the edges of the webs, of course, being held together by the rivets through the flange angles.

"Bent top flange angles of through girders should be spliced near the ends of the girder in order to permit of better handling in the shop. When this is not done, it often means that the long angles have to be swung across the shop, thereby interfering with other operations.

"Sometimes round-end girders are used when square-end ones would be satisfactory. The advantage of square ends, of course, is apparent. The round ends increase the shopwork very materially. The radius of round ends should not be less than 2′ 6", as the shopwork for round ends of small radii is not as satisfactory as that for those of large radii. For round-end girders with side plates, the latter should be cut where the bottom edge meets the curvature of the flange angle. The space under the flange angles beyond the side plates and above the web reinforcing plates is to be filled with fillers. This detail avoids the bending of the side plates or the use of plates with reentrant cuts.

"On skew spans the design should be studied with the object of squaring the ends of stringers, end frames, etc. When the skew is slight, it is often a very simple matter to square the end of the cross frame instead of using a skew frame.

"When trough sections or I-beams rest on the tops of columns, the milling of the said tops is not necessary, as the end shears from the troughs are carried by the cross-girders.

"For skew crossings, subways, etc., the amount of skew varies often by small amounts. The designer can simplify the work very materially by using the same amount of skew so that all cross-girders, trough sections, and I-beams may be made identical for the various crossings.

"Instead of using built-up bolsters which are uncertain in their workmanship because of the fitted stiffeners, use cast iron for light work and cast steel for heavier work. For complicated pin-bearing shoes, very good results can be obtained by using cast steel instead of building up with a large number of plates and stiffeners. All rail beds, where permissible, should be of cast steel. The top of the casting should be grooved to distribute the loads to the web of the casting.

"When designing the sections for top chords and end posts, the centre of gravity is above the middle of the member. The details, however, are simpler if the centres of pin-holes and working lines are designed to be in the middle of the member. It is not necessary to change the working line to the centre of gravity as the weight of the member will, to some extent, offset the effect of the unsymmetrical section.

"A common mistake in design is to proportion the members with too small a width, causing considerable trouble in packing the pins and in making room for the verticals, pin plates, etc. Another bad feature of narrow chords is that practically all rivets around the connections must be countersunk because of close space, and the ends of the posts must be cut away for clearance, thereby weakening the said ends. By adopting chords of larger widths much better details can be used around the pins at panel-points.

"The sizes of pins should be determined with the idea of using as few different diameters as possible; generally two or three sizes are sufficient for one span. It is not altogether a waste of material to use larger pins than necessary, as the bearing plates can be reduced in thickness.

"Eye-bars, adjustable members, turnbuckles, screw threads, segmental rollers, clevices, upsets, etc., should be designed according to the standards of the bridge manufacturers, for quicker deliveries and better fabrication are obtained by the use of standards.

"On cross-girders for subway crossings, the top and the bottom flanges of the girder are notched to clear the flanges of the columns to which they connect. The tendency is to fit the notch with too little clearance, and sometimes the cross-girders will not clear the rivet heads which project inside of the flanges of the columns.

"The cap plates on top of the columns should be left so as to be field-riveted, in order that the cross-girders may be dropped into place without spreading the column.

"When two or more truss spans are identical, or when they are similar and have the same field connections, the field holes should be reamed to an iron templet in place of reaming them while the members are assembled. This will facilitate the delivery of the work, and will make identical members throughout the structure interchangeable. The advantages in the field are evident, less time being spent in sorting and finding material.

"When the specifications call for material drilled from the solid on account of the use of alloy, high carbon, or very thick ordinary steel, the members should be designed with as few pieces as possible. Instead of using 5/8" or 3/4" plates, which generally are of the right thickness for punched work, the material should be ordered as thick as permissible within the mill requirements.

"NOTES ON BRIDGE ERECTING.

"In the designing of details extreme care should be exercised in arranging all joints and connections, so that the work cannot only be built at the shop for the least cost in labor and material, but also that it may be erected most economically and with a minimum of risk. In the case of bridgework, all connections should be so detailed that spans can be connected and made self-sustaining and safe in the shortest possible time.

"Unless for special reasons, it is usually customary to begin the erection of pin-connected spans at the centre panel, as this panel has adjustable members and the trusses can be squared up there before proceeding. The details should, therefore, be so arranged that the centre panel can be completed and made self-sustaining before the traveler is moved to the next panel. It is the usual custom for the erection to proceed from the centre panel toward the fixed end, and after this half of the span is erected, to proceed toward the roller end. Top chord sections in any particular panel are put in place after the posts and bars are erected; and it is especially desirable in heavy work that the details be so arranged that these chord sections can be lifted above the posts and set directly in place without being moved on end or sideways. Therefore, plates connecting two adjoining chord sections in heavy work should always be shipped loose.

"Wherever possible, in all truss spans the floor connections should be so arranged that the floor system can be put in place either before or after the trusses have been erected in their final position. It is usually customary, where local conditions will permit, to put the floor system in place first and erect the trusses afterward. This method of procedure has a great many advantages over that of raising the trusses first, viz.: there is a large saving in falsework, as longer panels can be used, putting bents directly under the panel-points and using the new floor system for carrying traffic and for running out material for the trusses; it permits the posts to be bolted to the floor-beams and released from the tackles on the travelers; it fixes the exact position of the shoes on the piers so that we can proceed with the erection from the centre either toward the fixed or the roller end, as we may prefer; it has the advantage of giving more opportunity for jacking up the spans in order to secure proper camber; and it requires a minimum amount of blocking. There are other features which render it desirable, where possible, to erect the floor system in advance of the trusses. Over dangerous streams, however, where there is a possibility of loss during the erection, it is sometimes desirable to erect the trusses first, so as to have as little material on the falsework as practicable and thus minimize the amount endangered. There are also sometimes certain local conditions which make it imperative that the trusses be erected first; and, therefore, it is important, wherever possible, that details be so arranged that either method can be used. In the erection of through, riveted, lattice spans, it is customary to place the floor system first, then to put the lower chords in position, set up the web members, and put the top chords on last. Therefore, it is more advantageous to have the gusset-plates connecting the web members with top chord riveted to the top chord sections rather than to posts or diagonals, as the rivets in gusset-plates connecting top chords with web members are more easily driven in the web members than in the top chord sections.

"In the case of through plate-girders, the details of the floor system should be so arranged that the stringers and floor-beams can be put in place, panel by panel, without the necessity of spreading the main girders. Particularly is this the case in 'Rolling Lift Bridges,' which, in the majority of cases, have to be erected in an upright position, and where it is extremely dangerous and practically an impossible operation to spread the trusses in order to put in place the floor system.

"Top chord sections with half pin-holes, having a hinge-plate on each section are undesirable. When half pin-holes are used, if possible put a hinge-plate on one section only and make it long enough to rivet or bolt to the adjoining section when in place. Hinge-plates should be arranged so as to give two full pin-holes in centre chord sections, and should be put on the ends farthest from the centre on the other sections, except in special cases when it is necessary to commence raising spans from the end instead of the centre.

"Entering connections are usually the most difficult and expensive to make; and where at all possible, entering connections of any character should be avoided, but where they must be used, particular attention should be given to insure necessary clearances. An entering connection is not only an expensive and dangerous operation, but in a great many cases it cannot be accomplished on account of the interference with back walls, adjoining spans, etc.

"It is of the greatest importance to allow ample clearance where members are packed inside of chords, posts, etc., as lack of proper clearance causes much trouble and expense, not only augmenting the cost of erection by increasing the time required for making the span safe, but adding materially to the risk. In putting in tie-bars and diagonals, it is customary to connect them on the bottom chord pins first, and then swing them into the chords and posts around the lower pins as a centre. All rivet heads coming in the path of bars swung in this way should be cleared. Too much attention cannot be given to this matter of proper clearance. Particularly is this the case in through and deck riveted lattice spans, which are being erected now more than ever before with the use of a derrick car with one boom; and the appliances for pulling tight-fitting members into place are not always present, as was the case formerly when these spans were erected by a gantry traveler. For adjustable rods packed close together, the sleeve nuts should be staggered. Attention should be given to the field connections so that enough space is allowed around all field rivets to enable them to be driven.

"All lateral bracing, hitch plates, rivet heads in laterals, etc., should be kept enough below the level of the top chords of girders, stringers, etc., so that the ties when in place will not foul them, it being an expensive operation to notch ties to clear such obstructions.

"Where laterals and hitch plates do not interfere with the loading of girders, and are not of such character as will allow them to be easily broken off, they should be riveted to the said girders; otherwise, they should be shipped loose or riveted to the braces.

"Particular attention should be given to the question of field riveting. Details should be closely examined with a view of minimizing the number of field rivets.

"It is not advisable to put two shoes on one bed-plate; but if this cannot be avoided, the bed-plate should be made longer and the anchor holes should be slotted to allow for variations in masonry.

"In the above suggestions of the points to be observed in order to facilitate the work of erection, the subject has been confined mostly to bridgework; the general principles, however, apply also to building work, particularly in the case of clearances, though entering connections, of course, cannot be avoided so much in building work as in bridgework.

"The following are the most important points to be observed in detailing, in order to facilitate and cheapen erection:

"(*a*) Avoid as far as possible entering connections.

"(*b*) See that proper clearances are given.

"(*c*) Minimize the number of field rivets.

"BRIDGE MACHINERY.

"When designing machinery it is quite common to model it after that in a former structure. When this is possible, design the machinery so as to use as many of the old patterns as can be utilized, thereby effecting an economy. It might be added that a repetition of patterns will also be of considerable help to the bridge shop. It is sometimes the case that variations are made without any advantage.

"The duties of each casting should be considered. For unimportant ones cast iron should be specified, and cast steel for those which are in bending or which carry large stresses. For a great many iron bases it is not necessary to plane the bottoms if resting on masonry. Steel castings, however, should be rough finished if resting on masonry. All castings which rest on other castings or on structural steel should, however, be finished.

"In designing structural work for movable bridges, sufficient clearance should be allowed between the movable and the fixed portions of the bridge. If possible, two inches should be provided for such clearance.

"Important holes in structural work connecting machinery should be reamed in the shop with the machinery assembled. Shims should be provided for vertical adjustment wherever required. If, in the judgment of the designer, it is extremely difficult to assemble the structural work in the shop, and if such assembling would not ensure perfect adjustment of the machinery, it is often advisable to note on the drawings that certain holes in the structural work connecting machinery are to be drilled in the field. Very often drilling in the field is much cheaper than assembling in the shop, and gives better results.

"For babbitt bearings, notches should be provided in the casting to hold the babbitt in place. These notches should be placed close to the point where the bearing is split so as to prevent the babbitt from springing up at the ends. For phosphor bronze bushings, the bushings should be held in place with short dowel pins. If the dowel pin penetrates through the bearing, it should be of brass. The castings for split bronze bushings should be ordered from the foundry as a full circle; for the clearance at the centre line will permit the machine shop to cut the bushing in half after finishing.

"For the racks of rim-bearing draw spans, finish the top surface so as to enable the machine shop to rotate the drum on the top of the rack while machining the top and bottom tread plates.

"Use cold rolled shafting for 3″ and under, as it gives very good results for light work. Shafting over 3″ in diameter should be forged and turned. When forged shafting is used, the portion required for bearings and gears only should be finished, the remainder being left rough as forged.

"As there is a great variety of specifications for special material, such as phosphor bronze, babbitt, cast steel, etc., the designer should embody these compositions for the various materials in his specifications or on his general drawings. Due consideration should be given to the conditions under which the special material is used. For instance, for a low-speed bearing, a babbitt should not be specified which is ordinarily used for high-speed bearings.

"On cast-steel bearings the bosses under the heads or nuts of bolts should be finished. This finishing, however, is not required when cast iron is used. For unimportant castings the designer should specify cast iron. Frequently cast iron is just as satisfactory for some details as cast steel would be. Cast steel should be designed somewhat differently from cast iron as regards clearances and rules for shrinkage. It should be remembered that due to unequal thicknesses the cast steel will be warped considerably; and this should be considered in allowing for clearances and for over-run and under-run.

"Sometimes a bearing composed of one-half babbitt and one-half bronze is specified. The portion which carries the load, of course, is the bronze part. This is unsatisfactory to obtain good results in the shop. It is better to specify an entire bearing of bronze.

"Considerable confusion has been caused in the past by having gear teeth figured to both circular and diametrical pitch. We would recommend that diametrical pitch be used in all cases by all designers, as the standard gear cutters on the market are based thereon.

"Segmental rollers for expansion ends should, preferably, be made of forgings, with sides forged smooth and parallel. We would recommend a standard size of these rollers to be 6″ finished diameter and 4″ wide. The teeth for holding these rollers in place should be of a standard size 2″ wide and 1″ thick. Use standard tap bolts for holding the keys and bars in place.

"Turned bolts should be employed for cases where the bolt is in shear or where it carries vibration, as in the case of bearing boxes. The turned bolts should be 1/32″ smaller than the size of the reamed or drilled hole, which should be given in sixteenths, and the size of the thread and nut should be 1/32 smaller than the diameter of the bolt, which from the sizes given herewith will always make the sizes of thread and nut in even eighths or quarters. To illustrate this, if the designer specifies a $\frac{7}{8}$ bolt, the size of the reamed or drilled hole will be $\frac{15}{16}$, the diameter of the bolt will be $\frac{29}{32}$, and the size of thread and nut will be $\frac{7}{8}$ diameter.

"Steel castings which carry bending stress should be annealed. It is not necessary to anneal steel castings used for shaft bearings or those which carry direct compression only.

"An important feature in machine designing is a design with the parts arranged for accessibility in making repairs. The designer should bear in mind that if any one piece breaks, the broken casting should be replaced without dismembering the entire machinery.

" VIADUCTS.

"When the track is on a grade and the grade is not very steep, make the two bents of one tower alike by adding filler plates on the tops of the up-grade columns. For steep grades make the two bents of one tower the same length, but square the longitudinal bracing. A good detailer will then make the punching the same on all four columns of the tower, but the gusset plates which connect to the longitudinal bracing will be different for the two bents.

"The tower girders should be made of the same depth as the long girders. This will simplify the detail by the omission of the brackets sometimes used on the tops of the columns, and will permit of better shop details; as web plates, stiffeners, etc., can be duplicated in spacing from one girder to another.

"When designing columns for the towers, the splices should be indicated so that the column sections should, preferably, be under 40′ long, but should not under any circumstances exceed 60′.

"Where two deck girder spans are adjacent, the end cross frames are placed close to the ends of the girders. The cross frames can not be set by swinging in from the end of the span. They must be erected by swinging in from the centre of the span. The stiffener angles carrying the cross girders should be built with the back of the angles toward the centre of the span.

"Instead of designing a structural base for the tower columns to rest on the masonry, a simpler and more efficient design is to transmit the bearing through a casting. The base of the column will be held to the casting by one or two pairs of connection angles. The base-plate should be omitted as being superfluous in this design."

These communications of Mr. Wolfel and Mr. Reichmann state very clearly and thoroughly the case from the manufacturers' point of view; and their suggestions should prove of great value to all designing engineers who specialize in structural steelwork. The author can endorse heartily almost all that both of these gentlemen state; but there are a few points on which, as a consulting engineer, he must very respectfully take issue, although, as before indicated, he recognizes that in their line they stand as the acknowledged authorities and that he cannot in any way claim to be an expert on shopwork, knowing about modern practice therein only the general features and details that have come to his attention from his experience as a large purchaser, for his clients, of structural steel.

The author believes firmly in sub-punching and reaming practically all riveted work that is not drilled from the solid metal; for experiments that he has made have proved to him that the punching not only hardens the metal around the hole but also starts incipient cracks therein, and that reaming removes most of the injured metal; consequently, in all cases where strength is either directly or indirectly of importance, he would adhere to the sub-punching and reaming. For instance, lacing bars in the past have not been considered of much importance; but nowadays they are often proportioned for computed stress, the amount thereof, however, being determined by very crude methods. It is important to have lacing, latticing, and stay plates as strong proportionately as the rest of the metalwork. There are a few parts of bridges, such as filling plates and stiffening diaphragms, where strength is of very little importance; and in these it would be perfectly proper to punch the holes full size.

Similarly, in the case of sheared edges, if the metal near the shear is to be depended upon for strength, the said edges should be planed, but, otherwise, the planing should be omitted. It is more than probable that the shearing of edges is just as destructive as the punching; for the brutality of the treatment of the metal in the two cases is of the same character and apparently of the same severity.

While it is undoubtedly difficult to procure a tight fit for stiffeners on rolled I-beams, it does not appear to the author as safe to omit them at the ends of railway girders that are supported from beneath, or even from those used for carrying heavy highway loads to the masonry, because the unsupported webs are not of a shape to resist satisfactorily severe pounding.

Although it is true that turning the flanges of channels in makes the riveting somewhat more difficult, it need not prevent the use of power riveters, except in the case of a few rivets; while it facilitates greatly the detailing by bringing the webs of main truss members in the same plane for the attachment of the gusset plates. Most of the author's riveted bridges are built in this way.

Again, it is important to have the batten plates inside of the gussets, and to carry them to near the ends of the member, both of which conditions the turned-in channels favor. Moreover, they generally involve an economy of weight of metal for lacing and battens. But one of the most important advantages of turned-in channels is that they permit the ends of the floor-beams to fit closely to the bottom chords without cutting either the chord or the beam, which is not practicable if the flanges of the bottom chords turn out.

If in viaduct construction the tops of columns are cut off so as to let the main girders be supported directly thereon, Mr. Wolfel's insistence on making the girder depth uniform from end to end of structure is justified; but in the author's practice the columns are carried up to the level of the deck, and both the longitudinal girders and the tower cross-girders abut into them; hence there is no objecting to making the comparatively short longitudinal girders over the towers shallower than the long intermediate longitudinal girders. This layout looks much better, and the corner brackets afford an excellent connection for the diagonals of the longitudinal bracing.

As explained at length in Chapter XVI, there is an objection to placing the end or pedestal pin of a riveted truss span below the bottom chord which is seldom considered, viz., that the thrust of a braked train, acting with a lever arm equal to the vertical distance between the centre of the chord and the centre of the pin, produces a rather large bending moment that has to be resisted by the stiffness of the bottom chord and that of the inclined end post.

The author believes that invariably the end floor-beams should rivet to the end posts of the trusses so as to make the lower lateral system a complete and harmonious whole, and it matters not if the end posts be inclined. Moreover, one of the great advantages of end floor-beams riveted to inclined end posts is that this method of connection tends greatly to stiffen the ends of the said posts.

The author in other chapters has expressed his objection to the use of single angles in tension.

He disagrees with Mr. Reichmann about the correctness of the policy of locating top chord pins anywhere except on the gravity line of the sections. It takes a very small eccentricity to produce a high intensity of bending stress on the chord. Again, it is not right to assume that the reverse bending moment due to the weight of the member between panel-points will counteract the bending moment due to the eccentricity; because the form taken under loading by the centre line of the long strut will be a waved line passing through the centre of the chord pins, being concave upward in one panel and convex upward in the adjoining one.

On general principles the author tries to bar out all cast iron from his bridges, even from the machinery, fearing that, if it be permitted in one place, the contractor will insist on putting it into another. Cast iron is nearly always inferior to cast steel for any purpose.

The author never could see the philosophy in the insertion of a casting between the foot of a column and the masonry. Such detailing is not conducive to rigidity, which is one of the main essentials in trestle and elevated railroad work, nor is there apparently any economy in weight or cost of metal involved. Moreover, it is almost a physical impossibility to place the base casting so perfectly on the masonry and to plane the adjacent surfaces of column and base so accurately as to obtain anything like uniform contact over the whole area of the column foot. The pressure is sure to concentrate at points and along edges, thus producing large bending moments on the column and an unequal distribution of pressure upon the masonry beneath the base.

CHAPTER XVIII

CLASSES OF TRAFFIC AND PROVISION THEREFOR

Bridges are built to take care of different kinds of traffic, either separate or combined. Those which provide for more than one kind have been termed by the profession "combined bridges"; but, unfortunately, the general public appears to experience considerable difficulty in distinguishing between that term and "combination bridges," or structures in which the trusses, as well as some other parts, are built of both wood and metal. It is with the "combined" bridges that this chapter deals.

The various kinds of traffic that pass over bridges are as follows:

First. Steam railway traffic.

Second. Street railway or electric railway traffic.

Third. Vehicular traffic.

Fourth. Herded animal traffic.

Fifth. Pedestrian traffic.

Structures that carry the first and second kinds are denominated "railway" bridges; those that are confined to the third, fourth, and fifth kinds are known as "highway" bridges; and those that support both railway and highway traffic are called "combined" bridges.

As herded animals are always driven over the wagon roadway, they need but little consideration except when that roadway is occupied at times by other than vehicular traffic, in which case provision must be made either for their passing such other traffic on the structure or for withholding either that traffic or the animals until the roadway is free.

In country bridges and those built in small towns pedestrians have to pass over on the wagon way; but in city bridges sidewalks or footwalks are provided for their use.

As a rule, bridges for carrying both railway and highway traffic are located in or near large cities, although an occasional structure of this kind is found in country districts. The principal advantage of this type of bridge is the saving in first cost, and its principal disadvantage is a reluctance to cross over it on the part of timid drivers, whose horses may be frightened by the trains. The saving in first cost of a combined railway and highway bridge, as compared with two separate bridges for railway and highway traffic, is considerable, because the piers for the combined bridge are but little, if any, more expensive than those for the railway bridge, and because the extra metal for the superstructure of the former in comparison with that of the latter is very much less in weight than the metal required for a separate highway bridge. The prejudice against combined bridges on account of danger is almost wholly unfounded, for horses soon become accustomed to railway trains, and, when screens are employed to hide the latter, but little trouble is experienced on account of frightened animals. These screens may be made either slatted or solid, the former offering less resistance to the wind, and the latter being the cheaper.

The advent of the electric railway has somewhat complicated the question of designing combined bridges, for now it is often necessary to accommodate all kinds of traffic on the same structure.

Combined bridges may be divided into the following classes:

1. Structures having a single deck for all kinds of traffic, the railway occupying the centre of the bridge, and the electric railway lying close to one truss.

2. Structures having a single-track railway at the middle, a narrow footwalk on each side of same inside of the trusses, and cantilever brackets outside of the latter to carry wagonways and electric lines. This arrangement may be varied by running the electric cars over the main railway track, thus leaving the wings free for vehicular traffic.

3. Structures having a double-track railway inside of the trusses, with long cantilever brackets outside carrying wagons and electric lines next to the trusses and pedestrians outside. This arrangement may be varied, as in Case 2, by carrying the electric trains on either one or both of the main railway tracks.

4. Structures having a double-track railway inside of the trusses, with short, cantilever brackets for wagon and electric-railway traffic outside, and either a single passageway overhead at the middle for pedestrians, or two passageways for same on overhead brackets outside of the trusses. As before, this arrangement may be modified by running the electric trains over the main railway tracks.

5. Double-deck, single-track structures carrying a railway train on one deck and vehicles and pedestrians on the other. If electric cars also are carried, they should generally use the railway track on account of the narrowness of the bridge; but by putting the railway below and using cantilever brackets above, the electric cars may share the wagonway and run over either one or two tracks. When the electric cars and the vehicles occupy jointly the upper deck, it is generally best to carry the pedestrians by cantilever brackets on the lower deck, as the structure might be too narrow to warrant caring for them above by footwalks outside of the joint wagon and electric car roadway, and because permitting them to use the said joint roadway would be too hazardous.

6. Double-deck, double-track structures carrying railway trains on one deck and vehicles, electric trains, and pedestrians on the other, or with the electric trains using the steam railway tracks. The vehicles and electric trains may either occupy the same roadway, or the former may be

carried on cantilever brackets, leaving the middle portion of the deck for the latter. In such a bridge the footwalks should be on cantilever brackets, either above or below, outside of the other roadways.

In double-deck structures where the steam railroad is below, it is necessary to use every precaution for keeping the locomotive fumes away from the upper deck, as smoke rising through the floor frightens horses even more than does the train itself. Moreover, smoke is exceedingly disagreeable to everybody passing over the structure. Again, the question of protecting the highway floor from being set on fire by sparks from locomotives must be satisfactorily solved in this combination.

Class No. 1 is the cheapest possible kind of combined bridge, and at the same time the most unsatisfactory, for when a railroad train is about to pass over the structure all vehicular and electric-railway travel must be kept off, and because pedestrians must look out sharply for their safety when on the deck with a railway train crossing. Their danger is really greater, though, when an electric train is passing a team or teams. The least allowable clear width of bridge for this class of structure is twenty feet, the electric cars running on a third rail and on one of the rails of the main railway.

Class No. 2 is a very satisfactory type of structure. The author has designed and built several bridges of this kind, the largest of which is the Combination Bridge Company's Bridge over the Missouri River at Sioux City, Iowa. It consists of two draw-spans of 470 feet each and two fixed spans of 500 feet each, besides the deck approach spans, the distance between central planes of trusses being twenty-five (25) feet.

Class No. 3 is also a satisfactory type of structure. The author once built a large bridge of this type, viz., the one across the Missouri River at East Omaha, Nebraska. This class of structure involves very heavy metalwork; but it is not uneconomical.

Class No. 4 is an unusual type, and is not likely to be called for very often, although the author has had occasion to figure on bridges of this kind.

Class No. 5 gives a satisfactory distribution of traffic, as was proved by the author's bridge over the Fraser River at New Westminster, British Columbia. In this the steam railway and the electric cars occupy a single track on the lower deck; and vehicles and pedestrians use in common a sixteen (16) foot clear roadway on the upper deck.

Early in 1908 in preparing a design for a combined bridge to carry a railway, a street-railway, wagons, and pedestrians over the Second Narrows of Burrard Inlet at Vancouver, British Columbia, the author evolved a rather novel method of dividing the traffic. The bridge was to be built at first to carry only the railway and the street-railway, but provision was to be made to take care of wagon and pedestrian traffic in the future. The distance between central planes of trusses being restricted from motives of economy to the least consistent with the Dominion Government's requirements for clear roadway—in this case nineteen (19) feet—it would have been improper construction to put twelve (12) foot roadways outside of the trusses and six (6) foot sidewalks outside of these; for such an arrangement would make each cantilevered portion of the deck wider than the distance between trusses, while good practice does not permit it to exceed two-thirds thereof. As the clearance above high water was ample on account of there being an overhead crossing of the Canadian Pacific Railway tracks at the south end of the structure, it was suggested to suspend the footwalks from the cantilever brackets that carry the roadways. This would necessitate small roofs to protect pedestrians from the roadway drippings. The arrangement described was shown by cost estimates to be exceedingly economical, but it was objected to on account of its interfering with the running of certain small craft under the swing span.

Class No. 6 represents a very good arrangement which can be modified to suit nearly any conditions of combined traffic. A good example of this type is the author's bridge over the Missouri River at Kansas City, Mo., owned by the Union Depot, Bridge, and Terminal Railroad Company, and known as the Fratt Bridge.

In designing combined bridges of all classes except No. 1, a considerable economy of metal may be effected legitimately by keeping the total live load for trusses as low as is proper with reference to the theory of probabilities. For instance, in Class No. 2 or Class No. 5 the live load for trusses may be determined by adding to the equivalent uniform live load, given by the diagram in Fig. 6*e*, a much smaller highway floor load per lineal foot of span than that prescribed in the specifications; because when the greatest train load is on the bridge, the chance of having simultaneously a heavy highway live load is very small. The longer the span the smaller may the live load per square foot of floor be taken when finding the total live load for the trusses. Again, in Classes No. 3 and No. 4 it would be legitimate to take the truss live load per lineal foot for the railway equal to twice the car load per lineal foot, and add thereto a small highway live load as in the last case. Finally, in Class No. 6 in case of a four-track bridge with cantilevered highways and footwalks, it would be proper to assume the live load for the trusses equal to the sum of the car loads per lineal foot on the four tracks and ignore entirely the vehicular and pedestrian loadings; for the greatest probable live load from all class of loading would never exceed the said four car loads, unless, perchance, some of the assumed electric-train loads were shorter than the span length, in which case a small allowance for a simultaneous loading from the highways and footwalks should be included.

This reduction of live load, however, can readily be carried to extremes, as was the case in the first accepted design of the great cantilever bridge over the St. Lawrence River near Quebec, and as is likely to be the case whenever the preparation of the specifications for a bridge is left either directly or indirectly to the contractor who is to build the structure. Good judgment, uninfluenced in any way by considerations of personal gain or by motives of false economy, should rule in the establishment of the live loads for the trusses of "combined" bridges.

CHAPTER XIX

FLOORS AND FLOOR SYSTEMS

THE floor is that part of a bridge which is supported on the steelwork (or on reinforced concrete girders) and carries the live loads directly. In a railway span it consists of the rails, guard-rails, and the ties in open-deck construction; and the rails, ties, ballast, and supporting deck for ballast in solid-deck construction. In a highway span it consists of the timber flooring, or the pavement with its base, and in addition the rails and ties in case a railway of any kind crosses the bridge.

The floor system is that part of the structure which supports the floor and transfers the loads to the trusses, main girders, or arches. It consists of the stringers and floor-beams in ordinary spans, and in addition occasionally the cantilever beams, which are merely the extensions of the floor-beams outside of the trusses.

While a structure usually has a floor and a floor system, it sometimes happens that it may have one without the other. In the case of a railway deck-girder span the floor is generally supported directly on the girders without an intervening floor system. This is also done in some of the shorter, single-track railway, deck-truss spans; and in through-girder spans the ties are sometimes supported on shelf angles riveted to the girder-webs, in which case no floor system is employed. Where the rails are carried directly on cross-beams with intervening plates but without ties, the rails and running-plates alone constitute what might be termed the floor.

As was shown in Chapter XVIII, bridges are designed for various classes of traffic, either singly or combined. A structure may carry a steam railway, an electric railway, or highway traffic alone; or it may carry a combination of these. Even as to highway traffic alone, one or more classes thereof may utilize the structure. After the division and distribution of the classes of traffic have been decided upon for any given bridge, the floor and the floor system should be laid out so as to carry these in the most economical manner practicable.

In railway bridges two main classes of floors are found, viz., the open and the solid decks; and in each class various types of construction are employed Each type has special features that make it particularly suitable under certain conditions, although it is practically always the case that various arrangements can be adopted for any given structure. When such is true, the particular choice will usually depend on the designing engineer or on the railway company. Both of the general classes are resorted to in structures with either limited or unlimited underneath clearances.

For ordinary construction where the underneath clearance is not limited, the open-deck floor of ties supported on stringers or girders is still generally employed. This is due largely to the lower first cost of the floor itself as well as that of the supporting metalwork, especially in the longer spans. It is a question, however, whether the adoption of the open deck is along the line of truest economy; and this question will have to be settled for any particular case as it arises. In spite of its increased first cost, many engineers are advocating the ballasted floor in place of the open deck, and are going so far as to say that the open construction in time will be entirely superseded. The eminent authority, A. F. Robinson, Esq., C.E., Bridge Engineer of the Santa Fé System, in a paper on ballasted bridge floors, published in the *Journal* of the Western Society of Engineers, Volume 10, makes the following statement as one of the reasons for using the solid type of floor.

> "This floor can be defended on financial grounds from the maintenance standpoint entirely, without taking into account the increased comfort to traffic, the increased life of the structure, and the decreased liability to damage in case of derailment.
>
> "Nearly all metal bridges will cost for lining and surfacing of the track something like 20 cents to 30 cents per ft. of track per annum. Ordinary track in ballast costs from 6 cents to 10 cents per ft. per annum for line and surface. The difference between these two items, varying from 14 cents to 20 cents, will pay five per cent on a little more than the difference in cost per lineal foot of this floor over the ordinary decks."

But aside from this matter of economy, there are the many other advantages of the ballasted floor. In the first place it provides a smooth-riding track, practically as perfect as a well-maintained track on the rest of the line, so that the change in train-motion in passing from the ordinary roadbed to the bridge is unnoticeable. It reduces the vibration in a structure, lessening the effect of impact and practically doing away with the noise caused by the train in crossing. This reduction tends to increase both the life of the bridge and the comfort of the passengers. Moreover, a greater sense of security is produced by such a structure in the minds of the traveling public, the trainmen, and the railroad officials as well. Derailments on ballasted structures have proved far less serious than those on bridges with open floors. Ties in ballast have practically no tendency to bunch or be otherwise displaced by a derailed train; and besides, the ballast gives a good support to such a train in addition to having a greater tendency to check its speed. Moreover, much less time is required to put the track in running order again, as it is generally only necessary to readjust the ballast and track and perhaps replace a tie or two, or possibly a rail.

A ballasted floor, unless supported on timber, is practically fireproof, and thus it not only saves expense for insurance and for replacements necessitated by fires, but also prevents disastrous wrecks and their consequent litigations and adjustments. Again, the usual deterioration of the steelwork caused by brine drippings from refrigerator cars is overcome by the use of a ballasted floor, provided the supporting deck be watertight, as it always should be. Moreover, a structure with such a floor will be found more stable in the time of high floods.

The alignment and surfacing of track constitute another feature favorable to the ballasted floor, and especially is this true in structures located on curves. Not only is difficulty experienced in the original construction of an open deck on such a structure but far greater trouble is met in maintaining the alignment and surface.

Again, where ballasted floors are employed, the poorer grades of timber can be used for ties. This is particularly worth considering in view of the fact that the better grades are rapidly becoming less plentiful and more difficult to obtain, and, therefore, more expensive.

The danger due to fires caused by falling coals might be overcome very easily in the ordinary timber floor construction by covering the deck with light sheet iron, as is done by certain railroads. This, of course, adds to the cost of the deck.

While many engineers advocate the ballasted floor on all of their bridgework, railroad companies, as a rule, do not employ it except where special conditions either warrant or compel its adoption. In almost all track-elevation work in cities, necessitated by the elimination of grade crossings, the ballasted floor is used. The requirement that the overhead structure shall be noiseless and shall give proper protection to the traffic beneath permits of practically no other construction. Then, too, this type of floor is much more rigid and has a considerably longer life than some of the other types of shallow floors that have been tried. It may happen at times that it is difficult to get away from one of these other types of floors; but, as a rule, they can be avoided. Where subways are frequent, a few inches of increase in the grade might have an appreciable effect on the kind of floor that can be used, while the increase in cost of embankments will be negligible as against the advantages gained by the ballasted floor. In certain localities solid floors are not required, although it may be necessary to provide a shallow floor. In such cases a substantial open construction can be resorted to.

As has been stated before, where there are no limitations set for the floor system, the usual type of floor consists of ties separated five or six inches and supported on stringers in the case of through spans and in the deck spans in which the trusses are spaced over thirteen feet centres, and on girders or trusses in the case of short, single-track, deck spans. Where ballasted construction is adopted, as a rule no provision need be made for taking care of the drainage. Usually in such a case a solid flooring of creosoted timbers is laid on the stringers, girders, or trusses;

and this is covered with the ballast in which the track ties are laid. A good description of a floor of this type, which was used in the reconstruction of the Atchison, Topeka, and Santa Fé Ry. Bridge over the Missouri River at Sibley, Mo., is to be found in *Engineering News* for Dec. 16, 1915. Other types of construction can be used to advantage on account of their longer life, although their cost is greater. Their construction is similar to that employed in the various types of floors for track elevation in cities, where they are generally adapted to through-girder or truss spans and where it is necessary to drain them properly. As their construction has largely developed with the track-elevation work where shallow, solid decks are necessary, their description will be taken up in connection with the discussion of shallow floors.

Where the latter are required, the through-girder span is the most common type of structure employed, as such shallow construction is generally found only in cities and in crossings over other roads where long spans are both uneconomical and unnecessary. This is not always true, however, as certain crossings will not permit of other than long spans, in which case through trusses with special floor construction have to be resorted to in spite of the fact that they are uneconomical.

In track-elevation work and in overhead crossings where thin, solid floors are demanded, many types have been devised. Of these might be mentioned the various kinds of trough floors which were among the first to enter the field. These were built up of different shapes and sections, and each had its good and its bad points. The rectangular trough built of plates and angles found the most general favor, as it was unpatented, easily varied to carry any loading, and readily constructed. However, the evolution of the thin floor had only begun, and these floors have since been largely superseded by later designs. An extended discussion of the trough floors can be found in the *Transactions* of the American Society of Civil Engineers, Vol. XXVII, in a paper entitled "Thin Floors for Bridges," by A. F. Robinson, Esq. They are again discussed in a paper on "Ballasted Bridge Floors," by the same author in Vol. 10 of the *Journal* of the Western Society of Civil Engineers, along with the other types of floors which have practically superseded them. The latter article shows in detail the various styles of floors considered when planning the track elevation in Chicago; and these are typical of the bridge floors used throughout the country. A comparison is made of these as to weight, distance from top of rail to clearance, number of rivets (both shop and field) required per lineal foot of floor, and cost.

The floors considered in this comparison consisted of several types of the trough floor and a series of floors built up of closely spaced I-beams placed transversely between girders or stringers and supporting various types of decks, some being ballasted and others not. One deck consisted of merely a steel plate covering the I-beams, in addition to the rails, which were placed directly on running-plates, no ties being used. In a modification of this deck, channels replaced the running-plates, and the rails were supported directly on timber sleepers resting in the channels. A further modification consisted in the use of ballast on top of the deck plate, in which the ties were laid. The type of floor adopted for certain of the track-elevation work was similar to the one last described, except that the deck plate was replaced by a flooring of creosoted tongued-and-grooved timber on which the ballast was placed. This floor gave good service and was extensively used. These floors are well described and illustrated in the article previously mentioned; and the reader is referred to it for complete details.

The latest trend, however, has been toward reinforced concrete construction; and it is safe to say that this will eventually completely replace all previously used types. It lends itself to the best advantage for various conditions, and can be constructed in numerous forms. Its weight and the consequent extra metal in the superstructure might cause the adoption of another type of solid floor for certain situations; however, the economy of this is questionable, and it is certain that for short spans it affords the best solution of the problem.

One of the lightest of this type of floors is similar to those previously mentioned, except that the deck, supported on closely spaced transverse steel beams, consists of a thin reinforced concrete slab which carries the ballast for the track. The beams may be spaced farther apart by increasing adequately their size and by augmenting the thickness of the slab. In another type of construction the transverse steel beams are spaced closely together and entirely encased in concrete, the thickness of the slab being such that the flanges of the beams are properly protected. No reinforcement is used in this construction, the beams being figured to carry the entire load, the concrete merely providing a protection for the metal and a deck to support the ballast. This construction is especially adapted to I-beam and short plate-girder spans. Where it is impossible to use any of these types of construction on account of limited vertical space, a specially shallow floor can be built by constructing reinforced concrete troughs between the beams and filling these with ballast for the ties. The bottoms of the troughs form short reinforced slabs resting on the flanges of the beams, while the sides form protection for the beams and can be reinforced to act with them in carrying the load to the girders. I-beam spans can be built of similar construction, except that the troughs have to run longitudinally and be wide enough to receive short ties for each rail. The troughs must be braced together transversely. Various types of ballasted reinforced concrete floors are described and illustrated in the article by Mr. Robinson in the *Journal* of the Western Society of Engineers previously referred to, and also in Skinner's well known work, "Details of Bridge Construction, Part III, Specifications and Standards." The American Society of Civil Engineers published a valuable paper, "Waterproofing Railroad

Bridge Floors," by Samuel Tobias Wagner, Esq., C.E., in its *Transactions*, Vol. LXXIX, which gives details of various types of reinforced concrete floors.

In deck-girder spans where solid floors are required, various types of construction are resorted to, among which are those previously described. In addition, however, the reinforced concrete slab supported directly on the girders and carrying the ballast for the track has been used extensively. These slabs in certain cases are cast in place and in others are built in sections on the ground or at some construction yard and set on the girders by means of derrick cars. This type of construction is also well illustrated and described in the articles previously mentioned. It can be adapted to truss spans where ordinary floor systems are employed, but where a solid deck must be used.

In order to prevent the water that falls on a solid floor from flowing through to the roadway below, it is necessary to waterproof the deck in some manner and to provide a means for removing the water quickly. Waterproofing should also be resorted to when the stringers are to be encased in concrete, for otherwise water may reach the steel and cause it to corrode, in addition to causing the encasement to crack and split off. Various methods of doing this have been employed for the different types of construction; and there is a great diversity of opinion among the many engineers designing such structures as to the best way in which to obtain the desired result. This can be plainly seen from the views presented in the before-mentioned paper by Mr. Wagner and in the discussions thereon. This is no more than might be expected, as the waterproofing of floors is still more or less in its infancy, having received its impetus with the track-elevation work at present under way. The American Railway Engineering Association also has made extensive investigations along this line, and its conclusions are given in its 1914 *Proceedings*. Prof. Milo S. Ketchum, in his monumental work, the "Structural Engineers' Handbook," on page 178 *et seq.*, also treats of the subject of waterproofing bridge floors.

In almost all construction the majority of engineers agree that the membrane method of waterproofing is the proper one to use for bridge floors; but as to the make-up of this membrane and its application there is a difference of opinion. Some consider an asphaltic mastic covering to be ample protection; others hold that a two-ply or three-ply mat of felt, burlap, or cotton fabric is sufficient; while still others maintain that a five-ply mat should be employed. No doubt the last-mentioned method is best; and it should be used on all important bridges that require waterproof floors.

In selecting the fabric for the mat, only such material as can be thoroughly impregnated with a protective fluid should be considered. Some engineers claim that this cannot be done with burlap or even with cotton fabric; and, therefore, they maintain that only felt should be employed.

However, it is a fact that a great deal of burlap is being used on important work; and it should not be rejected without thorough investigation. The mat is built up as it is placed. The burlap or felt should be well lapped at the edges, and the different layers should be thoroughly bonded with the waterproofing material. Asphalt or an improved waterproofing pitch should be used. The mat as a whole should not be bonded to the deck. It should be carried up at the sides above the highest possible level to which the water can rise.

Deck structures are easily waterproofed; but this is not true of through, plate-girder spans. The difficulty arising in this case lies in the construction at the web of the main girder. On account of the vibration in the span, the web usually separates from the deck, leaving a space for water to seep through. The only way to overcome this is to form a pocket in the concrete adjacent to the web, and fill it with pure asphalt having a low melting point. This will remain plastic under all temperatures, and will form a thorough seal between the steelwork and the deck. In such construction a concrete parapet should be built along the webs extending well above the ballast; and the waterproofing should be carried up with it. The mat at this point should be protected with a reinforced concrete covering extended up under flashing angles on the girders. Where the distance from the base of rail to the top of the girder is small, it is advisable to encase the entire upper portion in concrete and carry the waterproofing and protection up over the top flange.

The waterproofing mat under the ballast should be protected with either an asphaltic mastic about 1½ inches thick, a thin layer of reinforced concrete not less than 2½ inches thick, or a layer of brick. If bricks are used, the openings between them can be filled with the waterproofing pitch if the deck is level; but if the structure is on a grade, cement grout should be employed as a filler. The bricks should be placed in a mortar bed not less than ½ inch thick.

As it is imperative to remove the water promptly, the floor should be well drained. A slope of not less than one per cent is necessary to secure good results. For short structures, the water can be carried to the ends and removed back of the abutments. In this case the inclination can best be made in the steelwork, the necessary slope being provided either from one end of the span to the other or from the centre to each end. In removing the water at the abutments, care must be taken to provide ample drains behind, so that it will not flow back over the parapets onto the bridge seats. Such drains must provide against freezing in the winter, which tendency will cause the water to flow over the abutment parapets. As a rule, however, it will be best not to let the water drain off, back of the parapets, but preferably to remove it through drain pipes carried down in front of or through the abutments, the latter arrangement being adopted so as to protect against freezing.

Drain pipes under the floor are generally a nuisance to maintain; and they should not be employed when they can be avoided. It is usually difficult to give them proper inclinations; and unless protected, they will freeze in the winter time. Moreover, open drains are not satisfactory, as they furnish a place for birds to nest, which condition makes them difficult to keep in order. On long structures, however, some form of drain must be used; and the situation should be studied carefully in order to secure good results. The drains should be frequent; and each should be provided with a trap just below the floor so as to catch sediment and permit cleaning out. The water should be carried to the ground as soon as possible after entering the drains; and downspouts should be employed frequently to obtain this result. They should be of ample size and should be securely attached to the supports for the main girders.

In locations where it is necessary to use a thin floor although a solid deck is not required, various types can be resorted to. One that has been used to some extent consists of a floor system of closely spaced cross-beams carrying running-plates on which the rails rest directly. These running-plates provide for the attachment of the rails and give rigidity to the floor system. The main objection to this floor is the noise that accompanies the passage of trains. There is also danger in case of derailment due to the lack of a proper bed for supporting the derailed cars. Another type consists of a floor system of closely spaced cross-beams with short stringers riveted between them, each set of stringers supporting one tie. This construction is well adapted to structures on curves when it is necessary to superelevate the outer rail. It is, however, more costly than the previously described type, although it does not possess the disadvantages mentioned. Where space permits, the cross-beams can be spaced farther apart so that the stringers can be made to carry more than one tie. Still another type of shallow open floor has been used—that in which the ties are supported on shelf angles riveted to the webs of through-girders or where they rest on the bottom flanges themselves.

This last construction is far from desirable and should not be resorted to, especially in the case when the ties rest directly on the flanges. In the first place the ties for such a design have to be unusually large in order to carry the heavy wheel concentrations of modern live loads over the span between the girders. In the second place it is a very difficult matter to insert the ties between the girders even when the span is first erected at a time when no traffic interferes with the construction. In fact, it is necessary to saw the ends of the ties with special bevels so as to make it possible to enter them between the webs of the girders; and at the stiffeners, further precautions have to be taken. In the third place the springing of the ties under live load causes the webs of the girders to bend back and forth, with a consequent weakening thereof. This effect is much greater where the ties are laid directly on the flange angles, as it is impossible to support their outstanding legs as can be done when shelf angles are used. Again, with the ties supported on the flange angles, it is difficult to provide proper struts between the girders from which to stiffen the top flanges. It is always possible in such a situation to use some other type of shallow floor that will give far more satisfactory results. Even in the old days when live loads were light this type of floor was deemed objectionable by bridge experts; but it was used somewhat on account of the prevailing desire of general managers of railroads to cut down the first cost of new construction to an absolute minimum. Today, however, there is no excuse for such detailing even on pioneer roads. The author can recall but two occasions in his forty years of practice when he was guilty of using this pseudo-economic floor, but there may have been other instances which he has forgotten.

Where a floor system for open-deck construction is employed, it consists of stringers riveted to floor-beams. The floor-beams are riveted to the trusses or girders, as the case may be. End floor-beams should always be used, and stringer brackets should be provided between the end floor-beams of adjacent spans, and generally between the end floor-beams of the end spans and the parapets of the abutments. For solid deck construction in which a very shallow floor is required, the floor system usually consists of beams or troughs riveted to the girders or the trusses. In the case of truss spans these beams are connected either to the chords or to longitudinal beams suspended below them. When a solid deck is adopted, but the use of a shallow floor is not necessary, stringers and floor-beams are generally employed. The troughs or cross-beams are then riveted between the stringers or supported on top of them. This construction is also used in half-through girder spans.

As a general rule, only two stringers per track are employed, although four stringers are not uncommon. A shallower floor at greater expense can be obtained in this way, but the reason for its adoption is more a matter of providing for the safe passage of derailed trains. Some roads go to the extent of using the ordinary two-stringer construction and placing an extra stringer of normal section at the centre of the track and one on each side of the main stringers. Other roads have different practices along the same line.

In a skew structure the deck should always be squared at the ends. It is practically impossible to provide a smooth-riding track where the ties at the joint are supported partly on the bridge and partly on the filled approach. Moreover, it is impossible to maintain such a joint in even a fair condition. The abutment can be squared on top for each track separately, and short stringers can be extended to the skew end floor-beams.

The preceding dissertation refers largely to bridge floors for steam railways. However it also covers those for electric railways, although for the latter the open-deck construction is used almost exclusively.

The deck of a railway structure, as well as other features of the floor and floor system, is often determined by specifications or details furnished to the engineer by his client. It is, therefore, necessary to study such specifications carefully before determining the make-up of the deck and the layout of the floor system. The specifications in Chapter LXXVIII cover many points of this nature which will not be taken up in this chapter.

Although the standard gauge for most railway tracks, both steam and electric, is 4′ 8½″ on tangent, a different gauge is used in some sections of this country, especially for electric railways. Some foreign countries also employ different gauges; hence it is necessary for the engineer to secure this information before preparing his design, as it will affect the layout of the floor system. On curves the gauge has to be slightly increased. The *Manual* of the American Railway Engineering Association gives the following rule:

> "Curves eight degrees and under should be standard gauge. Gauge should be widened one-eighth inch for each two degrees or fraction thereof over eight degrees, to a maximum of 4′ 9¼″ for tracks of standard gauge. Gauge, including widening due to wear, should never exceed 4′ 9½″.
>
> "Where frogs occur on the inside of curves the gauge at the frog should be standard, or the flangeway of the frog should be widened to compensate for the increased gauge."

For electric railway spans it is usually sufficient to widen the gauge ¼ inch for all curves. This increase is to be effected by shifting the outer rail and keeping the inner one to true alignment. It should be made in the length of the spiral, in case a spiral is used, or in a rail length in advance of the curve in case no spiral is employed.

While the tracks are generally spaced thirteen (13) feet centres for steam railways and ten (10) feet centres for electric railways, certain sections of the country use different spacings, especially for electric railways. Under special conditions it might be advantageous or necessary to employ some other spacing. The Government of the Dominion of Canada specifies a spacing of fourteen feet.

As explained at length in Chapter VIII, while the amount of the superelevation of the outer rail is generally fixed by the specifications, there are conditions where superelevation is unnecessary and others where it is wholly impracticable, so that the specifications should not be followed blindly. Where for any reason the train invariably takes a curve at very slow speed, it is unnecessary to provide superelevation; and where turnouts from either a single or a double track occur, it is impossible to do so. Again, it might happen that a greater superelevation should be provided than that called for by the specifications for general practice. This could occur at places where the locomotive engineers might be tempted to break regulations and run at high speed so as to surmount a steep grade. The superelevation should be worked out in the length of the spiral or on the tangents at the ends of the curve, with a rise of not more than one (1) inch in forty (40) feet. The inner rail should be kept at grade. Fig. 8*a* gives the superelevation to be employed according to the specifications in Chapter LXXVIII.

Where the track has an appreciable change in grade on the structure, it is necessary to connect the two grades by a vertical curve. It is most convenient to break the grade over a pier or column where this can be done, as it makes the laying out of the steelwork easier and more satisfactory. It is not always possible, however, to do this, and hence the layout should be studied very carefully so as to obtain the best results possible with the fewest variations in the floor system. A parabolic curve should be used; and it is generally sufficient to extend it over one or two panels on each side of the break in grade.

The size and section of the rails for both steam and electric railways are usually fixed by the owner or determined by the engineer for any particular structure. The author generally uses the sections adopted by the American Society of Civil Engineers and prefers a section not less than seventy (70) pounds per yard for electric railways and eighty (80) pounds per yard for steam railways. For the heavier railway loads the rails should be of ninety (90) pound and even one hundred (100) pound sections. On very sharp curves, especially for electric railways, steel guards should be provided on the inside of the inner rail. These may consist of rails spiked to the ties, plates attached to the rails with filler blocks between the two, or a special guard-rail section. Thirty-three (33) foot rail lengths are generally adopted for steam railways and sixty (60) foot lengths for electric railways, although the thirty-three (33) foot lengths are sometimes used for the latter. The rails are generally connected with standard, six (6) bolt, angle splice-bars. As a rule, they are laid with broken joints, and with full openings between the ends at −20 degrees and none at + 120 degrees Fahrenheit, the temperature being that of the rail and not that of the atmosphere. On electric railways it is necessary to bond together the rails in each line, and to cross-bond the two lines of rails at intervals of not more than five hundred (500) feet. The author in his practice has used compressed terminal bonds, similar to Bond No. 7193 of the Ohio Brass Co., with 7/8″ terminals and 4 − 0 cable placed under the angle-bars at each joint. The bonds should be properly compressed into freshly drilled holes in the webs of the rails. Care should be taken to see that the rails are not connected in any way to the steelwork; and proper insulation must be provided where necessary. This is also required on steam railway bridges where there is employed any kind of signal system in which the rails form part of the circuit.

At the ends of movable bridges where it is necessary to break the rails, many devices have been employed to make the track continuous when the span is in position for traffic. Such devices are often standard with any particular railroad company. The author has used almost exclusively the gap-bar shown in Fig. 19*a*, and has found it very simple and satisfactory. The bar is made of manganese steel in order to prevent rapid wear. It is necessary that it be well supported, as shown. The same detail was adopted for the expansion ends of the fixed spans of the Pacific Highway Bridge over the Columbia River, designed by the author's firm. In this case the expansion ends of the adjacent fixed spans were placed on the same pier, which caused a large expansion to take place at that point. The same gap-bar can be used for the ordinary layout of spans, although it has been the general practice to make the rails continuous over the entire structure.

To overcome the effect of the expansion of the steelwork, a very good arrangement of the track was used by the Pittsburgh and Lake Erie Railroad on the Beaver Bridge over the Ohio River. It was described in *Engineering News* of January 26, 1911. The rails are supported on tie-plates fastened to the ties with screw-spikes. The heads of the spikes are seated on bosses on the tie-plates and extend over the rail flange with a clearance, however, of one thirty-second ($\frac{1}{32}$) of an inch between the spike heads and the rail flanges. This gives the rails a perfect freedom of movement longitudinally. At the fixed ends of the spans they are permanently attached to the deck by anti-creeping devices. The expansion and contraction of the spans and of the rails are, consequently, independent of each other.

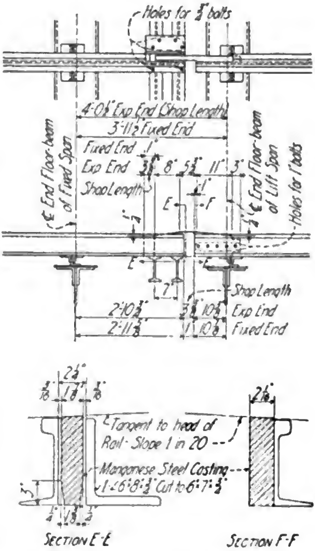

Holes for ⅝" bolts
4'-0⅛" Exp End (Shop Length)
3'-11½ Fixed End
Fixed End 1"
Exp End 3⅛" 8" 5½" 11" 3"
Shop Length
1"
E
F
℄ End Floor-beam of Fixed Span
℄ End Floor-beam of Lift Span
Holes for 1" bolts
E
7"
Shop Length
2'-10⅜" 3⅝ 10⅛ Exp End
2'-11⅝" 1 10⅛ Fixed End
2⅛"
$\frac{3}{16}$" 1⅞" $\frac{5}{16}$"
2⅛"
℄ Tangent to head of Rail - Slope 1 in 20
Manganese Steel Casting
1-∠6"·8"·¾" Cut to 6"·7"·⅝"
¼" ⅞" ¾"
Section E-E
Section F-F

Fig. 19*a*. Rail-gap Bar at Ends of Movable Bridges.

Where the track is broken at the ends of movable spans, the rails on both sides of the break should be firmly fastened to the deck by anti-creeping devices. A good detail of such a device is illustrated in the article just referred to. It consists of specially-made, wide-flanged plates bolted to the webs of the rails as well as to a wide, flat plate on which the rails rest. To this plate are riveted lug angles that bear against the ties; and the plate is also bolted to the ties or fastened thereto by screw-spikes. Lug angles to bear against the ties are riveted to the stringers, to which the ties are hook-bolted. This device has proved very satisfactory. On the fixed spans back of the break in the rails, switch points should be inserted in order to allow the rails to creep without disturbing the gap bars. If this is not done, great annoyance will be occasioned in the operation of the movable span by the interference of the rails of the fixed spans.

The dimensions of ties for track on tangent are usually given by the

TABLE 19*a*

SIZES OF TIES FOR VARIOUS RAILWAY LOADINGS AND SPAN LENGTHS. EXTREME FIBRE STRESS 2,000 LBS. PER SQ. IN. IMPACT 110%. LOAD DISTRIBUTED OVER THREE TIES. TIES ARE GIVEN WIDTH BY DEPTH.

Span Lengths in Feet C. to C. Stringers	Sizes of Ties in Inches							Span Lengths in Feet C. to C. Stringers
	Class 40	Class 45	Class 50	Class 55	Class 60	Class 65	Class 70	
6.0	7× 9, 8× 8	7× 9, 8× 8	7× 9, 8× 8	7× 9, 8× 8	7× 9, 8×10	8×10, 10× 8	8×10, 10× 8	6.0
6.5	7× 9, 10× 8	8×10, 10× 8	8×10, 12× 8	8×10, 12× 8	8×10, 10×10	8×12, 10×10	8×12, 10×10	6.5
7.0	8×10, 10×10	8×10, 10×10	8×12, 10×10	8×12, 10×10	8×12, 12×10	8×12, 12×10	10×12, 12×12	7.0
7.5	8×12, 10×10	8×12, 10×10	8×12, 12×10	10×12, 12×10	10×12, 12×12	10×12, 12×12	10×14, 12×12	7.5
8.0	8×12, 12×10	10×12, 12×10	10×12, 14×10	10×12, 12×12	10×14, 12×12	10×14, 12×12	10×14, 14×12	8.0
8.5	10×12, 14×10	10×12, 14×10	10×14, 12×12	10×14, 12×12	10×14, 14×12	10×14, 14×12	10×16, 12×14	8.5
9.0	10×12, 14×10	10×14, 12×12	10×14, 12×12	10×14, 14×12	10×16, 12×14	10×16, 12×14	10×16, 14×14	9.0
9.5	10×14, 12×12	10×14,	10×14, 14×12	10×16, 12×14	10×16, 12×14	10×16, 14×14	12×16, 14×14	9.5
10.0	10×14, 12×12	10×14, 14×12	10×16, 12×14	10×16, 12×14	10×16, 14×14	12×16, 14×14	12×16,	10.0
10.5	10×14, 14×12	10×16, 12×14	10×16, 12×14	10×16, 14×14	12×16,	12×16,	12×18, 14×16	10.5
11.0	10×16, 12×14	10×16, 12×14	12×16, 14×14	12×16,	12×16,	12×18, 14×16	12×18, 14×16	11.0
11.5	10×16, 12×14	10×16, 14×14	12×16, 14×14	12×16,	12×18, 14×16	12×18, 14×16	12×18,	11.5
12.0	10×16, 14×14	12×16, 14×14	12×16,	12×18, 14×16	12×18, 14×16	12×18,	12×20, 14×18	12.0
12.5	10×16, 14×14	12×16,	12×18, 14×16	12×18, 14×16	12×18,	12×20, 14×18	12×20, 14×18	12.5
13.0	12×16, 14×14	12×16,	12×18, 14×16	12×18, 14×18	14×18,	12×20, 14×18	12×20,	13.0
13.5	12×16,	12×18, 14×16	12×18, 14×16	12×18, 14×18	14×18,	12×20,	14×20,	13.5
14.0	12×16,	12×18, 14×16	12×18,	14×18,	14×18,	14×20,	14×20,	14.0

designing specifications or in the data furnished by the owner for any particular structure. On ballasted decks the regular track ties of the road with the ordinary spacings are generally adopted. For open decks, Table 19*a* shows the various sizes of ties for any given span and loading, while Table 19*b* gives the allowable span length for any size of tie and for any class of railway loading, in accordance with the specifications in Chapter LXXVIII. It must be remembered, however, that these ties are for the ordinary tangent alignment and for the usual dapping of one-half (½) inch. When it is necessary to dap the ties more than this amount and for structures on curves, special consideration must be given to their design. Specifications sometimes require the ties to be dapped so that one-half of the camber is taken out of the deck. The author does not follow this practice, as a rule; for he considers it more advantageous to make the ties uniform. Again, when ties are supported on girders with cover plates on the top flange, a variation in the dapping must be taken care of. The same thing holds true over the piers in deck spans on a skew. Where vertical curves are required, it is usually necessary to vary the dapping of the ties on the curve. In all of these cases the entire problem must be studied carefully in adopting any size of tie, and the strength of the tie at the support should not be overlooked. It is always

TABLE 19*b*

ALLOWABLE SPAN LENGTHS FOR TIES OF VARIOUS SIZES FOR VARIOUS RAILWAY LOADINGS. EXTREME FIBRE STRESS 2,000 LBS. PER SQ. IN. IMPACT 110%. LOAD DISTRIBUTED OVER THREE TIES.

Size		Moment of Resistance	Span Lengths in Feet C. to C. Stringers						
Nominal $b \times d$	Actual $b \times d$	$\frac{1{,}000\ bd^2}{3}$	Class 40	Class 45	Class 50	Class 55	Class 60	Class 65	Class 70
7× 9	6½× 8½	157,000	6.5	6.4	6.2	6.1	6.0	5.9	5.9
8× 8	7½× 7½	141,000	6.4	6.2	6.1	6.0	5.9	5.8	5.8
8×10	7½× 9½	226,000	7.3	7.0	6.8	6.6	6.5	6.4	6.3
8×12	7½×11½	330,000	8.2	7.9	7.6	7.4	7.2	7.0	6.9
10× 8	9½× 7½	178,000	6.8	6.6	6.4	6.3	6.2	6.1	6.0
10×10	9½× 9½	286,000	7.8	7.5	7.3	7.1	6.8	6.7	6.6
10×12	9½×11½	419,000	9.1	8.7	8.3	8.0	7.7	7.5	7.4
10×14	9½×13½	577,000	10.7	10.1	9.6	9.2	8.8	8.5	8.3
10×16	9½×15½	761,000	12.5	11.7	11.0	10.5	10.0	9.6	9.3
12× 8	11½× 7½	216,000	7.1	6.9	6.7	6.5	6.4	6.3	6.2
12×10	11½× 9½	346,000	8.4	8.0	7.7	7.5	7.3	7.1	6.9
12×12	11½×11½	507,000	10.1	9.5	9.0	8.7	8.4	8.1	7.9
12×14	11½×13½	699,000	11.9	11.1	10.5	10.0	9.6	9.2	8.9
12×16	11½×15½	922,000	14.1	13.1	12.3	11.6	11.1	10.6	10.2
12×18	11½×17½	1,174,000	16.6	15.4	14.3	13.5	12.8	12.2	11.6
12×20	11½×19½	1,458,000	19.5	17.9	16.6	15.5	14.6	13.9	13.3
14×10	13½× 9½	406,000	9.1	8.6	8.2	7.9	7.7	7.5	7.3
14×12	13½×11½	596,000	10.9	10.2	9.7	9.3	8.9	8.6	8.4
14×14	13½×13½	820,000	13.1	12.2	11.5	10.9	10.4	10.0	9.6
14×16	13½×15½	1,081,000	15.7	14.5	13.6	12.8	12.1	11.6	11.1
14×18	13½×17½	1,378,000	18.6	17.1	15.9	14.9	14.1	13.4	12.8
14×20	13½×19½	1,711,000	22.0	20.1	18.6	17.3	16.3	15.4	14.7

best to use the same sized tie with the same framing on any one structure; and the layout of the steelwork should be studied to gain this end, if possible, but, of course, not at the expense of the said steelwork.

For bridges on curves where no superelevation is required, it is necessary to figure the maximum moment on the ties and determine the section from Fig. 16*i*, except in the case of flat curves where the stringers are practically parallel and symmetrically spaced with respect to the rails, in which case the size of ties can be taken directly from Table 19*a*.

Where the track is superelevated, the ties have to be proportioned just as stated, but the question of the size of the tie will depend on the method adopted for taking care of the superelevation. Different methods are used. The most common and probably the best of these is to keep the stringers or girders in any cross-section at the same elevation and use a tie of constant depth with a different dap at the two ends. This, however, becomes uneconomical where the superelevation is large, and in such a case it is best to adopt beveled ties. In the author's practice ties with a depth of as much as twenty-two (22) inches at the deeper end have been used in deck-truss spans where they were supported directly on the chords. This, however, is extreme, and any particular case should be looked into carefully before adopting such a depth. Where it is practicable to do so, some engineers incline the stringers or girders so as to have their flanges in a plane as nearly parallel to the plane of the track as possible. This permits the use of a constant depth of tie without excessive dapping, but makes the layout of the steelwork, as well as the fabrication, more difficult. Another method that has been used to obtain a constant and shallow depth of tie is to keep the girders vertical and to make them of different depths. This, however, is more objectionable than the method just described. The author has never resorted to either of these practices, and does not consider their adoption advisable. A better method of providing for excessive superelevation is to use constant depth ties that will figure for the maximum moment, and attach beveled shims under the higher ends. These should be well fastened to the ties and should have sufficient depth over the support to prevent any tendency to break down under pounding. The shims should never be placed on top of the ties under the high rail; neither should longitudinal shims be placed on top of the stringers under the ties at the outside of the curve, although such shimming is permissible in highway bridges. In figuring the effects of track curvature Chapter VIII should be studied carefully.

On curves the ties are to be made symmetrical about the centre line of track as far as it is possible to do so. They should be spaced uniformly over the spans and laid at right angles to the centre line of the bridge, except near the ends of the spans where they are fanned out so that the space between the ties at the ends does not vary more than one (1) inch from the uniform spacing.

In all cases the length of the tie is to be such that the ends extend at least six (6) inches beyond the outside edges of the supports. The ties should always be kept clear of the flanges of the floor-beams so as to prevent the formation of a dirt pocket over them. Wherever the rail is as much as three (3) inches above the floor-beams, a shim of the same width as the floor-beam flanges should be laid on top of them. On the fills back of the abutments the length of the ties must be such that the guard-timbers that are extended out from the bridge can be fastened to them. The ties on the parapets should not be laid directly on the concrete but should be supported on short shims or blocks in line with the stringers or girders, which will hold them away from the concrete.

At the refuge bays on long deck structures three or four ties must be extended to support the plank flooring. At the outer end a substantial timber handrail should be provided. Walkways for the full length of the structure are sometimes called for by the owners for the use of the trackmen. In such cases every third or fourth tie should be extended so as to carry the walkway flooring in single-track railway spans; and in double-track spans every third or fourth tie should extend over both tracks, and the walkway should be placed between the tracks. On account of the danger from hot coals dropping from the locomotive, in no steam railway bridge should the walkway be placed along the centre of the track; but in electric railway structures this is permissible, although not advisable from the point of view of safety of pedestrians. A walkway from eighteen (18) to twenty-four (24) inches wide will be ample.

In open-deck railway-floors the life of the tie is not dependent to any great extent upon decay, but rather on mechanical wear and deterioration under loading. The best grades of timber are needed for such floors, and they should be thoroughly protected by the use of tie-plates of proper design. Also the question of rail fastenings should be carefully considered. In ballasted decks and on the approaches it is not necessary to use the best kinds of timber for the ties. In fact, the supply of this class of timber is so limited, when due consideration is given to the probable future demands, that the railroad companies for some years have been experimenting with the poorer varieties and have found certain of these to give excellent service when properly treated and protected from mechanical wear. These experiments are still under way both as to the species of timber that can be successfully employed and as to the proper methods of treating them. The reader is referred to the *Proceedings* of the American Railway Engineering Association for the most valuable information on this subject. It has been shown that these treated timbers are more economical for ordinary track construction than the best timbers untreated. In any case where it is deemed advisable to use treated ties, the treatment should be such as to protect the tie properly for its life as gauged by its resistance to mechanical destruction. It is necessary, however, to protect them from mechanical wear, as treated ties are more susceptible to the destruction caused by the pounding of the rails than are the untreated ones. The American Railway Engineering Association has made extensive investigations regarding tie-plates, as to their use and proper design; and the reader is referred to the Report of the Committee on Ties in the 1914 *Proceedings* for their most recent conclusions. These might be stated briefly as follows: Flat-bottom tie-plates become loose when cut-spikes are employed, resulting in mechanical wear; but when used with screw-spikes they have proved very successful. Plates with deep ribs or claws cut the ties so as to admit moisture, thus causing decay; and are, consequently, undesirable. Plates with cross ribs not over $^3/_{16}''$ deep are just as efficient as those with deeper ribs, even when used with cut-spikes; and they do not seriously damage the ties. The width of the tie-plate is an important element in determining the mechanical wear, and the use of plates less than seven (7) inches wide with soft wood ties will often determine their life. The plates should be of ample strength to distribute the load to the ties without deflection. Cut-spikes break down the structure of the wood, thus facilitating decay through moisture. This is avoided to a considerable extent where the spikes are driven in bored holes, and such spikes have about the same holding power as spikes ordinarily driven. Diamond-pointed cut-spikes are preferable to those with wedge points when thus driven. Where treated ties are used, all boring should be done previous to treatment.

On account of the unsatisfactory results in the life of the tie due to the use of the cut-spike, the screw-spike has received considerable attention in this country during the last few years. In Europe it has been used for a long time, and it was there that it received its greatest development and widest application. In France it is used almost exclusively; and the cut-spike is rapidly disappearing in the rest of Europe. In the United States some of the larger railroads have submitted the screw-spike to numerous and severe tests; and one of the latest articles on the results of a set of such experiments was published by the American Railway Engineering Association in the 1915 *Proceedings*. In that article G. J. Ray, Esq., Chief Engineer of the Delaware, Lackawanna, & Western Railroad, gives his reasons for adopting the screw-spike, the details of the spikes, linings, and tie-plates, the methods of preparing the ties and driving the spikes, the tests made, the conclusions reached, and the precautions to be taken. The results of the investigations cannot well be given here on account of the large amount of space they occupy. The article is full of valuable information throughout; and the reader can well afford to take the time to look it up, if he be at all interested in the matter. The investigations showed up very favorably for the screw-spike. A few of the many points brought out in the article might be mentioned. The question of whether it will pay to use screw-spikes will depend on the cost of ties, their probable life, and the amount of traffic. Tie-plates should always be used with screw-spikes, and they should have a shoulder on each side to prevent the rails from cutting into the spikes, as well as bosses to support the heads. The size of the spikes and the design of the thread should be carefully considered before its adoption, as a change later on would not permit the utilization of the old holes without damage to the wood fibre. The heads should have tapering sides so as to permit the use of a standard wrench in case of decreased size due to rusting. Where brine drippings have to be dealt with, the heads should be of ample size to provide for deterioration. All ties should be bored before treatment. The holes should be of proper dimensions for the class of wood used, with due regard to the size of the screw-spike. Holes should be bored somewhat deeper than the length of the screw-spike, and there is no objection to boring clear through the ties.

In connection with its many investigations along the lines of track betterment and economy, the American Railway Engineering Association has made many service tests of various forms and makes of ties designed to take the place of timber ties. So far the results have not been very satisfactory, except, possibly, in regard to the Carnegie steel ties, which have been used quite extensively on certain roads and have given good service. Further investigations are necessary in order to determine the advisability of adopting any of the many ties proposed. The latest report on the subject is to be found in the 1915 *Proceedings;* and on pages 536 and 537 of that book are given references to preceding volumes.

A single line of outside guard-timbers is commonly employed for railroad bridges; and inner guard-rails, preferably of steel, ought to be used as well. The latest conclusions of the American Railway Engineering Association regarding guard-timbers and guard-rails are as follows:

"(1) It is recommended as good practice to use guard-timbers on all open-floor bridges; and the same should be so constructed as properly to space the ties and hold them securely in their places.

"(2) It is recommended as good practice, in the installation of inner guard-rails, to extend them beyond the ends of the bridges for such distances as are required by local conditions, but that this distance, in any case, be not less than 50 feet; that guard-rails be fully spiked to every tie, and spliced at every joint; that the guard-rails be some form of metal section, and that the ends be beveled, bent down, or otherwise protected against direct impact with parts of the moving equipment.

"(3) It is recommended that the guard-timber and inner guard-rail, when used, be so spaced with reference to the track-rail that a derailed truck will strike the inner guard-rail without striking the guard-timber.

"(4) The inner guard-rail must not be higher nor over one inch lower than the running-rail.

"(5) It is recommended as good practice to use inner guard-rails on all open-floor bridges and on the outside tracks of all solid-floor bridges and similar structures longer than 20 feet on main-line tracks, and on similar bridges and structures on branch-line tracks on which the speed of trains is 20 miles per hour or more."

The discussions before the annual conventions which adopted the above conclusions will be found on page 1136 *et seq.* of Vol. 14, and page 1036 *et seq.* of Vol. 15, of the *Proceedings* of the American Railway Engineering Association. It will be noted that there was much difference of opinion regarding the last conclusion, many of the engineers considering its provisions to be somewhat too stringent. It is certainly, however, in the interest of first-class construction. A very satisfactory finish can be given to the inner guard-rails by bringing their ends together at the centre of the track, and connecting them with a cast nosing. Re-railing frogs should be placed on the approaches at short distances from the structure. The outside guard-rails should extend a like distance beyond the ends of the bridge; and the best practice would place a substantial pier at such points either to force the derailed cars toward the track or else to break a coupling-bar so as to prevent the cars from reaching the bridge. In ballasted deck structures substantial parapets at the sides are an important safeguard to derailed trains, and cost should not be considered in making them amply strong.

The standard practice has been to dap the outside guard-timbers over the ties to prevent bunching, and to secure them to every other tie by a three-quarter inch bolt; but the 1915 Report of the Committee on Wooden Bridges and Trestles of the American Railway Engineering Association (see Vol. 16, pages 893, 894, 1180, and 1181) suggested that it might be found better to fasten the guard-rails to every tie by lag-screws, and omit the dapping. The lag-screws should be screwed into bored holes, not driven in like spikes. More definite recommendations will doubtless be made in the 1916 Report of the same Committee.

In highway bridges various types of floors and floor systems are employed, and the design depends largely upon the location of the structure, the class or classes of traffic crossing it, and the funds available for its construction. As explained in Chapter XVIII, a structure may carry only one roadway over which all classes of traffic (pedestrian, vehicular, and even electric or steam railway) pass; but the steam railway should generally be separated from all other classes, if practicable. When a structure is of some importance, separate roadways are provided for each kind of traffic, with a possible combination of steam or electric railway traffic with that of light, fast, automobile traffic. In bridges located outside of cities, usually a single roadway is provided, although in more important bridges, where pedestrian traffic is heavy, a sidewalk or sidewalks should be employed. As a rule, steam railways do not cross such structures. City bridges generally have separate roadways for vehicular and pedestrian traffic; and where electric railways cross the structure, they usually occupy part of the main roadway. Sometimes separate roadways are employed for the heavy, slow, vehicular traffic and the light, fast traffic, especially where electric lines cross the structure, in which case they occupy the roadway for the light traffic. Steam railways may have a special deck or may occupy part of the roadway in the same manner as do the electric railways. The adoption of any particular traffic or combination of traffic will determine to a large extent the construction of the floor and floor system.

The lightest and most unsatisfactory construction consists of the old type of timber floor of wooden stringers covered with one layer of plank forming the roadway. This construction was not so objectionable when first employed, as the good hardwoods that were then easily procured had a fair life. Moreover, the traffic in those days was light, so that these floors did not wear out quickly. With the growing scarcity of good hardwoods and the increase in live loads, the maintenance of plank floors has become a difficult problem. With the soft woods and heavy traffic the flooring lasts but a short time—sometimes not more than a few months. On account of the lack of uniformity of the timber in the flooring, certain pieces will wear out and require repairs sooner than others. Such repairs usually consist of laying short planks over the worn-out parts of the floor, thus producing irregularities in the surface, which, when struck by the passing traffic, set up excessive vibrations in the steelwork. It is not infrequent that such repairs are not made until some animal has been seriously hurt by having its foot caught in one of the holes in the floor. In isolated districts where the traffic is light and timber plentiful, such construction is still permissible, but not in growing communities of any size.

Where the single plank floor on timber stringers is employed, the flooring, consisting of planks three (3) or four (4) inches thick, is usually laid diagonally on longitudinal timber joists supported on top of the floor-beams or on shelf angles attached to the floor-beam webs. This construction is generally adopted for through bridges, which are used almost exclusively for such locations. Where deck structures are built, the joists are placed at right angles to the girders, and the planking is laid on these, as before. Where the joists are placed on top of the floor-beams, the intermediate ones should be lapped over them with a small air space between. The outside joists, however, should be in line, on account of the handrail connection. These joists can be fastened to lugs riveted to the top flanges of the floor-beams. All such joists should be well cross-braced at the floor-beams and at intermediate points, especially in the case of the outer lines, because of the possible twisting effect of the handrails when the latter are attached to the outer joists.

A better construction in the plank floor is the use of steel stringers in place of the timber joists. The intermediate stringers should be I-beams and the outside ones channels with the flanges turned in so as to provide an easy connection for the handrails. The stringers should be riveted into the floor-beams and not supported on top, as the former method produces a much more rigid structure. Nailing-strips on top of the stringers should be used to provide a means for spiking down the floor. These can be fastened to the steel by bolts passing through the shims and through holes in the top flanges of the stringers. This, however, reduces the flange section of the beams in addition to weakening them otherwise. A better detail consists of clips fastened to the nailing-strips and passing under the top flanges of the stringers. This detail is shown in Fig. 19*b*.

Sometimes a floor is made up of two layers of planking, the base plank and the wearing surface. This construction is not, as a rule, adopted for the ordinary plank floor just described. The timber in such a floor should be well treated—at least the base plank and supporting shims or joists should be; but it is not convenient to use treated timber on small isolated structures. It has, however, a particular advantage in floors of combined bridges, where a special roadway is provided for steam or electric railway and light vehicular traffic to the exclusion of all other traffic. In this construction the rails, usually about five (5) inches high, are supported on wooden ties, which, in turn, rest on steel stringers riveted between the floor-beams. The size and spacing of the ties will depend on the loads and the spacing of the stringers. The same spacing of ties may be used throughout the cross-section, or a wider spacing may be allowable outside of the tracks. The more economical arrangement should be adopted. The lower or base plank is generally made about three (3) inches thick, leaving about two (2) inches for the wearing surface. The base plank should be laid longitudinally on the ties, and the wearing surface transversely between longitudinal timbers along the rails and under the guards at the sides of the roadway. This permits the replacing of the wearing floor without disturbing the rest of the deck. The wearing surface, including the strips along the rails, should, preferably, be of hardwood or else the best grade of long-leaf yellow pine, Oregon fir, or other suitable timber. It should not be treated, although all the rest of the timber should receive a thorough impregnation of creosote oil. The top surface of the base planks should be well swabbed with hot pitch before laying the wearing floor. The ties should be countersunk to receive the cup washers for the hook bolts so that no part of the bolt projects above the tie. This type of floor is shown in Fig. 19*c*.

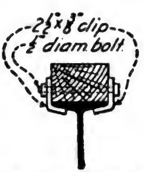
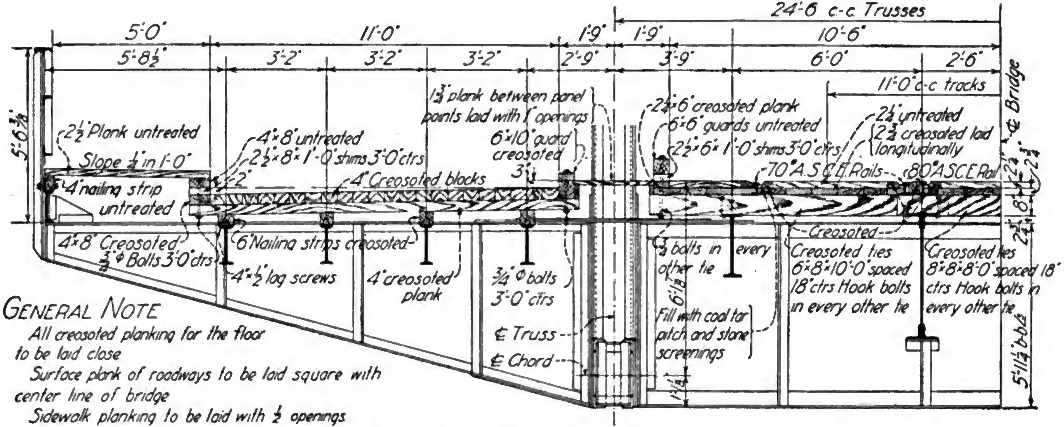


Fig. 19*b*. Clips for Timber Shims.

The construction last described is well adapted to bascule bridges where wood-block and other pavements cannot be used. Great care must be exercised in securing the flooring to the steelwork. The short life of such a floor under heavy traffic is, however, a serious drawback, and the City of Chicago has used a patented floor which has been quite successful. This consists of one (1) inch elm strips that have been dipped in asphaltum, placed on edge, and bolted together in sections containing from five (5) to ten (10) square feet.

Wood blocks are now used very extensively for pavements on bridge floors; and they make one of the most satisfactory roadways obtainable.


24'-6 c-c Trusses
5'-0"
11'-0"
1'-9"
1'-9"
10'-6"
5'-8½"
3'-2"
3'-2"
3'-2"
2'-9"
3'-9"
6'-0"
2'-6"
11'-0" c-c tracks
℄ Bridge
5'-6⅞"
2½" Plank untreated
Slope ½ in 1'-0"
4" nailing strip untreated
4"×8" Creosoted ¾" Φ Bolts 3'-0" ctrs
4"×8" untreated
2½"×8"×1'-0" shims 3'-0" ctrs
2"
4" Creosoted blocks
6" Nailing strips creosoted
4"×½" lag screws
4" creosoted plank
1¾" plank between panel points laid with ½" openings
6"×10" guard creosoted
3"
¾" Φ bolts 3'-0" ctrs
℄ Truss
℄ Chord
2¼"×6" creosoted plank
6"×6" guards untreated
2½"×6"×1'-0" shims 3'-0" ctrs
70# A.S.C.E. Rails
80# A.S.C.E. Rail
2¼" untreated
2¾" creosoted laid longitudinally
Creosoted
¾ bolts in every other tie
6'-1⅛"
Fill with coal tar pitch and stone screenings
1'-1⅛"
Creosoted ties 6"×8"×10'-0" spaced 18" ctrs Hook bolts in every other tie
Creosoted ties 8"×8"×8'-0" spaced 18" ctrs Hook bolts in every other tie
2¼"
2¾"
8"
2¼"
5'-11¾-6-6¾

GENERAL NOTE

All creosoted planking for the floor to be laid close

Surface plank of roadways to be laid square with center line of bridge

Sidewalk planking to be laid with ½ openings

Outside of Trusses two 8" spikes to be used at each shim for roadway plank

Between Trusses longitudinal plank to be fastened to each tie with two 5" spikes

Two 5" spikes to be used to each square ft of top course

Sidewalk planking to be fastened with two 5" spikes at each end

Fig. 19c. Cross-section of Floor of the Arkansas River Bridge at Ft. Smith, Ark.

They are light, wear well, and can be adapted to any grade. On steep grades they can be arranged so as to give the safest pavement for horses. The blocks, which are manufactured from suitable timber and thoroughly treated, vary in depth and width from three (3) to four (4) inches and in length from six (6) to ten (10) inches. The width should always differ from the depth so that the blocks will be laid with the grain up. Special blocks are necessary along the rails. The blocks are generally placed at right angles to the roadway, except at the sides where two or three rows should be made to follow the curbs. On grades of less than three (3) per cent, the blocks should be laid close but with sufficient openings between them to permit the proper application of the filler. On grades of more than three (3) per cent the blocks in transverse rows should be separated by creosoted laths about one and one-quarter ($1\frac{1}{4}$) inches wide and three-eighths ($\frac{3}{8}$) of an inch thick, placed on edge. The openings left between the rows of blocks above the lath are filled with gravel, not over one-quarter ($\frac{1}{4}$) inch in any dimension, from which the sand has been removed, or with stone chippings of equal grading. The spaces between the blocks in either the close or the open construction and the voids of the stone in the latter are filled with pitch to the top of the pavement, care being taken to see that the surface is left free therefrom, sand alone never being used as a filler. The blocks are then covered with small stone chippings to a depth of about one-half ($\frac{1}{2}$) inch and thoroughly rolled. This gives a good surface to the pavement and also takes up any pitch or creosote oil that comes to the top. Near each curb one or more pitch joints about one (1) inch wide should be provided to take care of expansion, the number depending upon the width of the pavement.

The wood blocks are laid on either a treated plank base or a concrete slab. When placed on a plank base, a layer or two of tar paper, thoroughly swabbed with hot pitch, is usually placed between the blocks and the plank. When a concrete base is used, various constructions are employed for bedding the blocks. One that has been used to a large extent in Europe, and which has been reported as giving excellent service, consists in finishing the concrete to the true crown of the roadway with a float and placing the blocks directly on this floated surface without any intervening cushion. In this country a sand cushion from one-half ($\frac{1}{2}$) to one (1) inch thick has been used extensively, although it has not been altogether successful. The sand has a tendency to flow as pressure is applied to one edge of a block, so that the blocks are not stable. This has been overcome to some extent by using a very thin cushion. On grades, however, a worse trouble has arisen. Here the sand has a tendency to work down grade, especially when wet; and during construction a rain may necessitate the replacing of a large area of blocks already down. To overcome this trouble as well as the previous one, a mixture of sand and cement has been frequently used for the cushion, and has given satisfaction. Some advocate using a dry mixture and placing the blocks on this; others prefer placing the cushion dry, but sprinkling it with water just in advance of laying the blocks; while still others recommend the employment of a slightly dampened mixture. Where the work progresses rapidly and the pavement can be rolled immediately after the blocks are laid, the latter method should give the most satisfactory results. Where this is not the case, the cushion may set before the pavement has been rolled, which is not advisable. The object in any case is to obtain a good, firm bed for the blocks, and the best way to secure this must be left to the judgment of the engineer on any particular piece of work. When it is necessary to waterproof the floor—and even when it is not—a very satisfactory bed is afforded by a waterproofing mat. This can be made of two-ply or three-ply burlap thoroughly lapped at the edges and bonded together with waterproofing pitch of a proven quality. Where the waterproofing is specially called for, the mat should be extended up the curb to the top of the pavement and keyed into the former. The method of waterproofing the floor of the Twelfth Street Trafficway at Kansas City, Mo., which structure was designed by the author's firm, is outlined in the specifications given in Chapter LXXIX.

Where plank is used as a base for wood-block pavement, it is supported on treated timber stringers, on steel stringers, or on treated ties which in turn are supported on steel stringers. The first two methods are particularly adapted to roadways without railway tracks, while the last is used almost exclusively for roadways that carry railways. The first method should be employed only where the cheapest construction is necessary. It is well suited to a deck structure, in which the joists can be placed transversely on top of the girders. In the second method the planks can be laid directly on the stringers and hook-bolted to them, or upon nailing-strips fastened to the stringers. The latter method is to be preferred, although the former is used by many engineers. The roadways outside of the trusses, which are illustrated in Fig. 19c, have flooring of this kind. In this type of construction the floor system can be built so as to provide for tracks. Mr. Thomas Ellis Brown, C. E., designed a very satisfactory detail for supporting the rails, as shown in Fig. 19d. A description of it is given in *Engineering News* of March 25, 1915.

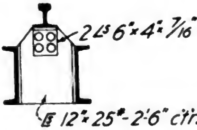


Fig. 19*d*. Rail Support

Where ties are employed, they should be arranged with the stringers to give the most economical layout.

A concrete base for a pavement is invariably supported on either steel or reinforced-concrete stringers or girders. It may be carried on crossbeams resting on longitudinal stringers, as was done by the author's firm on the Pacific Highway Bridge at Portland, Ore., shown in Fig. 19*e*. In this case a bitulithic pavement was adopted.

The size of the rails to use will depend on the type of construction employed. With the plank base, a seven (7) or eight (8) inch rail supported on the ties is generally required, its height depending on the depth of the timber. With such deep rails tie-bars should be employed to maintain the gauge of the track. Seven-eighths (⅞) inch bars spaced about six (6) feet centres and placed just above the base plank have proved satisfactory. For the construction used by Messrs. Boller, Hodge, and Baird, any size of rail can be adopted, as its support is independent of the floor. A rail about five (5) or six (6) inches high is to be preferred. When the blocks are supported on a concrete base, the rails may be carried on steel ties embedded in the concrete, in which case the said rails should be about five (5) inches high; or they may rest on cross-beams, as in the Pacific Highway Bridge, under which circumstances a much deeper rail will be required. Steel ties, when used, should be about 6′ 6″ long, placed about 2′ 0″ centres; and they should always be embedded in a plain concrete base which rests on the reinforced-concrete floor-slab.

Bitulithic and asphalt pavements on concrete bases are used quite extensively and have proved satisfactory in many cases. On steep grades they have a tendency to flow in warm weather, and in winter they become very slippery and difficult for horses to travel on. Local conditions should be carefully studied and specifications thoroughly drawn and rigidly adhered to in adopting either of these pavements.

Brick and stone-block pavements are not generally used for bridge floors on account of their excessive weight. They may be employed on short spans where the extra dead load will not materially affect the weight of the superstructure. Moreover, on the approaches they can be used to advantage. They should be laid on a one-and-a-half (1½) or two (2) inch sand-cushion overlying a concrete base; and the joints between the blocks should be filled with cement mortar. Transverse expansion joints are required about every twenty-five (25) feet along the roadway, and longitudinal joints are needed along the curb. These joints should be filled with pitch.

For small highway bridges with light traffic where a more permanent roadway is desired than the plank floor provides, but where a less cost than that for any of the floors previously mentioned is a necessity, a concrete pavement can be adopted. This consists of a concrete slab with an extra thickness for wearing surface. This extra thickness should not be less than three (3) inches. The slab and its reinforcement should be so designed that the wearing down of the surface will not affect its strength. The wearing surface should be made of a 1 : 2 : 3 mixture of cement, sand, and small broken stone. The slab is generally supported on steel stringers riveted to the floor-beams.

Buckle-plate floors are not now used to the extent that they have been in the past, as they have been replaced by other types. While they are very light, they are more expensive than the type shown in Fig. 19e, which is, moreover, a better floor. In very long spans and in cantilever

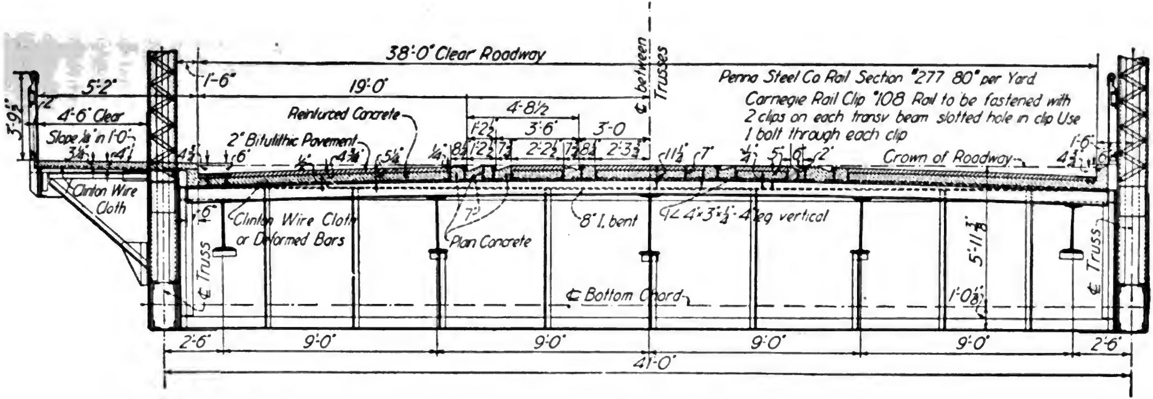


Fig. 19e. Cross-section of Floor for the Pacific Highway Bridge over the Columbia River

structures with large openings there might arise a case where the metal saved in the superstructure by the use of a buckle-plate floor would make up for the extra cost of the said floor. However, such structures are rare; and even then it is a question whether some other type of floor is not preferable, for quite lately it has been found that buckle plates are unsatisfactory for supporting block pavement, which the unavoidable deflections permit to be broken up under heavy concentrated loads.

Sidewalks are usually placed outside of the roadways. They are of either untreated timber or reinforced concrete on the bridge proper, or of concrete slabs on the approaches. They are supported on the inside by the curb and at intervals by timber or steel stringers. Where the main roadway lies entirely between the trusses, the sidewalks are generally carried on small cantilever brackets extended out from the trusses at the panel points. Where the main roadways lie outside of the trusses, the sidewalks are supported on extensions of the cantilever brackets that carry the roadways. The sidewalk flooring should be so thick that its strength will not be seriously impaired when wear takes place. Moreover, in case there is any likelihood of the traffic from the roadway getting on the sidewalk, it should be strong enough to take care of such unusual loads with a small margin for safety. The same is true also of the flooring over the spaces between the truss members, when the roadways are extended beyond the trusses.

On all structures it is necessary to take care of the drainage. With single plank flooring this is done by leaving one-half (½) inch spaces between the planks. On other floors it is necessary, however, to crown the roadways so as to carry the water to the curbs. A crown of one-sixtieth (1/60) of the width of the roadway will be ample for this purpose. The parabola, which has generally been used for laying out the cross-section of the roadway, gives too flat a surface for the centre portion. To overcome this, a broken curve has been used by many engineers. The general practice is to make the distance from the crown to the pavement at the quarter point from the centre one-eighth (⅛) of the total drop; at the half point, three-eighths (⅜); and at the three-quarter point, five-eighths (⅝). This can be easily obtained in the ordinary highway floor without tracks as well as in the floor with tracks where timber ties are not used. Where such ties are employed, the breaking of the floor as called for by this method of crowning may add a considerable amount of timber to the ties due to lapping them at the supports where the breaks are made. To obviate this difficulty the following methods can be employed. For a single track at the centre, this portion of the roadway can be made level, and that beyond the track can be given a uniform slope to the curb. For a double track, each track can be given an inclination corresponding to the centre slope in the method first described, and beyond the track the roadway can be inclined to the curb so as to provide the required crown. Sidewalks should be given a slope of about one-quarter (¼) inch to the foot toward the inside so that the water will drain onto the roadway.

On curves the crowning of the roadway is well adapted to the inner track for double-track structures as well as to a single track where it is not considered necessary to superelevate the outer rail. But the outside track, where a double track crosses the bridge, is at a decided disadvantage, as it has a reverse inclination to what it should have. This may not be objectionable where the curve is light or where the trains or cars take the curve with a moderate speed. However, where it is desirable to take the curve with a high speed, this is not permissible, and a remedy must be resorted to. Where a single track is placed at the centre of the structure, it can be given the proper superelevation by reducing the slope of the inner part of the roadway slightly and increasing it on the outside. If this gives excessive side slopes, it will be necessary to modify the floor construction so as to reduce them. In steam-railway work, when one-half (½) inch superelevation is sufficient, no trouble will occur if the above method is employed. The same is also true in double-track work, where it is necessary to continue the slope of the inner track past the outer in order to give it a proper superelevation. This puts the two tracks in the same plane. This raising of the floor at the outer track must be worked out beyond the ends of the curve either in the floor timbers or the floor system.

The drainage can be disposed of by letting the water escape under the guard-rails or through the curbs at the sides of the roadway, or the curb can be made solid and the water taken off by drains connected to openings in the floor. With the former method, care should be taken to see that the water does not drip on the steelwork. Where the latter method is employed, the drains should be of ample capacity to handle the water. The ordinary gratings used over openings in the floor are not satisfactory, as they choke up quickly, and as the usual care given to a bridge floor is not sufficient to keep them open. Basins with both roadway and curb inlets are very efficient, especially where they are properly designed. The openings should be large, and the basin should be arranged so that it will not retain any trash. In certain locations the water can be wasted under the structure; in others it is necessary to carry it off in drains to some convenient place. The drain pipes must be of sufficient capacity to remove the water quickly; and they should be firmly attached to the structure.

On bridges with level grades it is advisable to drain the rail grooves. One and one-quarter (1¼) inch wrought-iron pipes spaced about sixty (60) feet centres will take care of this drainage properly. The upper end of each pipe should be fitted with a flange to hold it in position; and the bottom should extend below the adjacent steelwork so that the water will not drip upon the metal.

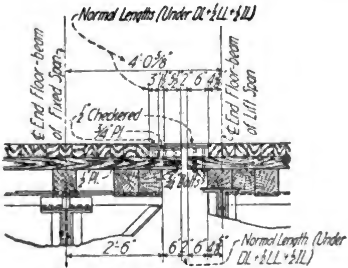
The curb should be of such a height as to confine the water properly and to give protection to the sidewalk. A six (6) inch curb is generally high enough, and in no case should the height exceed ten (10) inches. This should take care of any condition that may arise. Where the curb heights are large, excessive shimming for the sidewalk stringers may be necessary. It is advisable to protect the curbs against wear, especially on steep grades where there is a great tendency to brake the wagon wheels against them. Timber curbs and guards are best protected by steel angles attached to the upper inside corners. They should be fastened with No. 30 wood screws in countersunk slotted holes spaced about eighteen (18) inches from centre to centre. Angles can likewise be attached to concrete curbs, although there are certain curb bars on the market that are very good.

The grades on a structure should be given careful consideration in view of the traffic that is to cross it. Grades of two (2) and three (3) per cent are satisfactory for practically all classes of traffic. However, when the latter is exceeded, the grade becomes excessive for heavy traffic. Grades up to five (5) and even six (6) per cent are sometimes impossible to avoid, and the former is generally satisfactory for ordinary traffic. With the more general use of motor-trucks these steep grades can also handle the heavy traffic; but as long as the horse-drawn dray is in existence, there will always be a decided objection to them. Where a rather sharp curve is encountered on a steep grade, the inside of the roadway should be given the same degree of inclination as is used on the tangent, especially where a limiting grade has been established. In no case should any part of the roadway have a heavier grade than the traffic warrants. In fact, on curves in highway structures, it is even advisable to reduce the limit set for straight roadways as is done in railroad work.

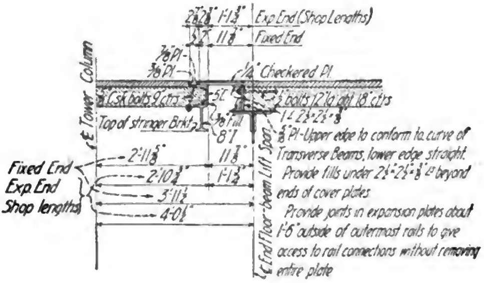
Where a change in grade occurs on the structure, it is necessary to ease the break by means of a vertical curve. The parabola is generally used for this; and it can be worked out in one or two panels on each side of the break. A curve fifty (50) feet long will ordinarily be sufficient.

At the free ends of the spans it is necessary to break the floor and to use expansion plates so as to provide a continuous roadway over the openings. These covers are best made of checkered or indented plates, usually riveted to base-plates. Such checkered and indented plates are not made in thicknesses greater than one-half (½) inch; and for this reason it is necessary to use compound plates, except where a one-half (½) inch plate will provide the requisite strength. Fig. 19*f* shows details of two types of expansion joints in roadways that have been used in the author's practice, one of them being for timber decks, and the other for a pavement resting on a concrete slab. In structures on grade the top plate should be placed on the up-grade side of the joint in order to keep the water from flowing under the plate. At the ends of draw-spans the floor cover-plates have to be lifted by a mechanical device in order to swing the span. This can be arranged so as to be operated by the same mechanism that actuates the end wedges and the gap bars for the rails.

The actual designing of the floor system will not be taken up completely in this chapter, as that portion of the structure is made up almost entirely of beams and girders, the main points in their design being covered


Normal Lengths (Under DL+½LL+½IL)
℄ End Floor-beam of Fixed Span
4'-0⅝"
3½ 3½ 2 6 4½
⅜ Checkered ⅜" Pl
℄ End Floor-beam of Lift Span
⅝ Pl
⅝ Bolts
2'-6"
6½ 6 4½
Normal Length (Under DL+½LL+½IL)

DETAIL FOR TIMBER DECK


2½ 2⅞ 1'-1⅜" Exp End (Shop Lengths)
3½ 2 11⅜" Fixed End
⅞ Pl
⅞ Pl
½" Checkered Pl
Csk bolts 9" ctrs
5E
⅞ bolts 12" lg alt 18" ctrs
L 2½×2½×⅜
Top of stringer Brkt
⅞ Fill
8⅞
℄ Tower Column
⅝ Pl-Upper edge to conform to curve of Transverse Beams, lower edge straight.
Provide fills under 2½×2½×⅜ L beyond ends of cover plates
Provide joints in expansion plates about 1'-6" outside of outermost rails to give access to rail connections without removing entire plate
Fixed End
Exp End
Shop lengths
2'-11⅜"
11⅜"
2'-10½"
1'-1⅜"
3'-11½"
4'-0½"
℄ End Floor-beam Lift Span

DETAIL FOR REINFORCED CONCRETE DECK

Fig. 19f. Roadway Expansion Plates.

fully in Chapter XXI, "Plate Girder and Rolled I-Beam Bridges." There are, however, a number of special features that need consideration, which will be discussed in this connection. The loads as well as special rules governing the design are given in the specifications of Chapter LXXVIII. The details for lateral and sway bracing connections will be found in Chapter XX.

Wherever it is possible to do so, the stringers should be placed below the top flanges of the floor-beams so as to obviate coping the stringers. This can usually be done in railway spans, but care must be taken to see that the rails will pass over the floor-beams. Then, too, sufficient space must be left below the stringers for the laterals. In through highway structures, with timber floors supported on ties or on steel stringers with nailing strips on top, the same detail can generally be worked out, although with wide roadways it may be necessary to cope the centre beams in the latter case or else use an excessive amount of shims. In deck structures it will usually be found impossible to avoid coping most of the stringers, as the floor must be supported on the girders or trusses. Care must be taken to see that the floor-beams or cantilever-beams do not interfere with the floor at the curbs, where it is usually the lowest. Where it can be done, the stringers should be placed entirely below the flange angles so that fillers will not be required on the cross girders. It might even be economical to cope the stringers so as to clear the vertical legs of the flanges. The stringers are usually made vertical, although in the construction shown in Fig. 19*e* it was considered advisable to incline them, as otherwise a large number of small bevelled shims would have been required on the stringers under the cross beams.

In through truss spans the end connection angles for the stringers should have outstanding legs not less than six (6) inches wide, with the rivets placed four and one half (4½) inches from the backs, so as to provide for the stretch in the bottom chord of the trusses under live load. In the end panels, however, the connection angles against the end floor-beams must have narrow outstanding legs if the end rivets take the moment from the stringer brackets. Where top strap-plates are used to connect the bracket to the stringer, the wide-legged angles can be used. Where the above moment is large, it is necessary to make the web and bottom flanges of the end stringers bear against the webs of the end floor-beams in order to take the thrust from the stringer brackets at the bottom and thus increase the lever arm of the resisting moment; and in such a case the brackets should be detailed to deliver their thrusts at that point.

The length of the stringers must be accurate, and to secure this result the end connection angles are either fitted to the stringers in frames or milled after they have been riveted to the beams. The former method is the usual practice followed by the shops. The thickness of the end angles should not be less than three-eighths (⅜) of an inch for highway work and seven-sixteenths (7/16) of an inch for railway work where they are fitted to the stringers. Where they are milled, these thicknesses should be increased one-sixteenth (1/16) of an inch. The rivets in the end connection angles through the webs of the stringers must be proportioned for the maximum end shear, while those through the cross-girder webs must be figured for the same load in shearing and for the greatest cross-girder concentration in bearing. At expansion points in the floor system of long spans it is usually difficult to drive rivets in the end connection angles in the field, so that it is advisable to provide driving-fit turned bolts at these places.

The rivet spacing in the webs of floor-beams should be arranged for multiple punching as far as it is possible to do so. The fillers under the stringer connections should be omitted, if the details can be thus worked out satisfactorily. When thin fillers—say ½″ or less in thickness—are used, and there is an excess of rivets through the stringer connection angles, they should be made of the same width as the said connection angles, and should be shop-riveted to the floor-beams with countersunk rivets placed in line with the stringer webs. When the fillers required are thick, or when there is no excess of rivets in the stringer connection, the said fillers should be made six (6) inches wider than the width of the connection angles, in order to provide for a line of shop rivets along each edge. This detail should also be used when it is possible to put enough rivets in the stringer connection angles to carry the end shear of one stringer but not enough in bearing on the floor-beam web to care for the floor-beam reaction. Where timber approaches connect to the main structure, shelf angles should be attached to the end floor-beams of the end spans in order to support the approach stringers; for otherwise it will be necessary to provide a special support for these on the end piers. Shelf angles should also be riveted to the floor-beams under each stringer for supporting the latter during erection. These angles should be of minimum sections; and rivets should be provided for the erection load only. No reliance should be placed on these rivets for transferring the stringer load to the floor-beam.

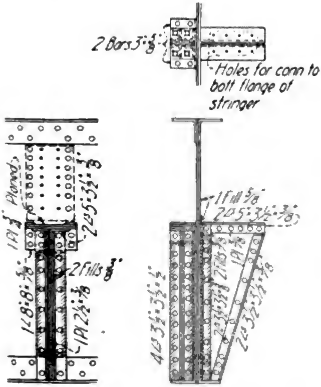


Fig. 19g.

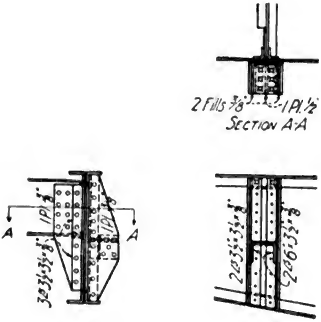


Fig. 19h.

Expansion Pockets for Stringers.

At expansion joints in the floor system, expansion pockets are required for the stringers. These are riveted to the cross girders; and they should be amply strong for transferring the stringer reactions to them, due allowance being made for the bending moment on the groups of rivets. Figs. 19*g* and 19*h* show typical expansion pockets used by the author. The detail shown in Fig. 19*g* is adopted where there is ample depth below the stringer to develop the pocket; and the detail illustrated in Fig. 19*h* is employed where this depth is limited.

The end connection angles for the floor-beams should conform to the requirements previously given for stringers as to thickness. In fixing the length of the outstanding legs it is necessary to consider the spacing of the rivets in the trusses and the number thereof that can be driven. In riveted trusses the end detail of the intermediate floor-beams is comparatively simple; but in pin-connected trusses it is often difficult to obtain a satisfactory detail on account of cutting out the girder to make room for the pin. Two conditions arise in the latter case—the one in which the end reaction can be taken care of in every respect in the depth of the girder above the cut, and the other in which it is necessary to transfer part of the said reaction to the trusses above the top flange. Where only a few additional field rivets are required, the outstanding legs of the top flanges can be cut back and the end connection angles extended sufficiently to secure these extra rivets. The section of the connection angles must be sufficient, however, to transfer the stress carried by these rivets. Where a satisfactory detail cannot be worked out in this way it is necessary to form a bracket by splicing the web and extending it above the top flange. In all cases, except where the cut is very shallow, reinforcing plates are required to strengthen the web; and they are frequently needed to develop the bottom flanges properly.

Where it is not necessary to extend the connection above the top flange, the vertical section along the cut should be figured, as this is the weakest part. Its net moment of inertia must be used, and the value of the plates and angles entering into it must be determined by the rivets available for developing their strength. The web reinforcing plates should be extended to such a point that the shear carried across this section can be transferred to the web. There should be sufficient rivets between the end of either flange and the first stringer to transfer the entire flange stress in this distance. If this requires a rivet pitch of less than two diameters, reinforcing plates extending over the web plates and the vertical legs of the flange angles are needed to relieve the horizontal section of the web next to the flange, which would otherwise be overstressed in shear. Beyond these patch plates the flange pitch should not be less than two diameters. The same procedure should be followed in floor-beams with brackets extending above the top flange. In this case, however, the flange stress on the upper inclined fibre, as determined by the usual method, must be multiplied by the square of the secant of the angle between the inclined edge and a normal to the section. Moreover, it is necessary to test the horizontal section of the bracket just above the top flange in a similar manner. The moment on this section is computed from the proportionate share of the end reaction carried by the rivets that are located above the top flange of the girder. Ordinarily, it will be best to cut off the outstanding legs of the top flange angles so as to permit the end connection angles to be continuous; for cutting the latter at this point would weaken the horizontal section just mentioned. Reinforcing plates are placed against the floor-beam web between the flanges, extending under the end connection angles; and they are usually carried back so as to act as splice plates for the web at the point where it is cut. Additional reinforcing plates can be placed over the flange angles, end connection angles, and splice plates, if such reinforcing be needed. A very good detail can be obtained in such a case by cutting back the vertical legs as well as the horizontal legs of the top flange angles, and adopting end angles of the same thickness as the flange angles, unless this would give excessively thick end angles. The latter are to be placed directly against the web. Reinforcing plates of the same thickness as the end angles are then placed between the said flanges; and outer reinforcing plates are added, extending over the first plates and the legs of the flange angles and end connection angles. Occasionally, in light work, it is possible to cut back the top flange angles, place the end connection angles against the floor-beam web, and not use either fillers or reinforcing plates, provided the web is ample for resisting both the bending moment and the shear at this point. If the section of the floor-beam at the point where the splice is located is sufficient to resist the bending moment without the aid of the reinforcing plates, the splice can be designed in the same manner as for any ordinary girder in so far as the details on the side next the centre of the floor-beam are concerned. In testing the strength of various sections, special care should be taken to see that to no part is there assigned a stress greater than can be developed by the rivets therein on both sides of the section. Sections for testing should be taken parallel or normal to the direction of the external forces acting thereon, for the ordinary assumptions regarding the distribution of stresses over a cross-section do not apply to inclined sections.

In deck structures where cantilever-beams outside of the girders are used, the tops of the floor-beams and those of the said cantilever-beams should be in the same plane and flush with the top cover plate of the main girder. This permits the strap plates to pass over the girders. At other panel-points it may be necessary to use fillers between the strap plates and the top of the girder. This detail necessitates cutting back the flanges and the top corners of the webs of the cross-girders; and in so doing they should be trimmed in such a way that fillers will not be required against the webs of the main girders.

In through, plate-girder spans the top flanges thereof have to be supported laterally from the floor-beams. The best detail consists in splicing the web and extending the part next to the girder above the top flange in the form of a bracket. Stiffeners should be used at all panel-points, shop riveted to the girders; and the cross girders should be field riveted to their outstanding legs. The brackets need not always extend quite all the way to the top flange; but in any case they must be deep enough to develop the end reaction. Preferably, though, these brackets should extend clear up to the top flange. The bracket plate should be tested at a horizontal section just above the top flange for the proportionate part of the load transferred by the rivets in the bracket. If this section figures weak, additional end connection angles should be provided on the floor-beam proper opposite the stiffener angles. The flange angles on the side of the web against the stiffeners of the main girder must be cut back to clear them; but on the other side they should extend the full length of the beams. These details are illustrated in Fig. 21*x*.

Where cantilever beams are used, it is necessary to connect them to the floor-beams by strap plates at the top; and thrust angles are required at the bottom, unless both bear against the same web plate as in ordinary girder construction, in which case the thrust angles are not needed. In through truss spans the tension straps must pass outside of the posts and be connected to the floor-beams and cantilever beams through horizontal connection plates; while in deck-girder spans they can be connected directly to the top flanges of the cross beams. This can also be done sometimes in the end cantilever beams of through truss spans, and in deck truss spans, if the tops of the cross girders and cantilevers are placed at the same elevations as the tops of the upper chords. Should the total thickness of such strap plates equal or exceed three-quarters ($\frac{3}{4}$) of an inch, it will be economical to use two or more plates. The top plate should engage enough rivets at each end to develop it properly, and each succeeding plate should extend far enough beyond the one above it to develop it fully. In truss and through-plate-girder construction it will frequently be necessary to slot gusset plates or girder webs to provide for the passage of the tension straps. The arrangement of the thrust angles is likely to vary greatly; and in the case of end floor-beams it may be necessary to make the shoe take care of the thrust from the bottom flange of the cantilever beam. Where this is done, it will be well to have the holes in one end of the tension straps drilled in the field, or at least sub-punched in the shop and then reamed in the field; for it is practically impossible to secure a satisfactory matching of parts at such a point in any other way. In some cases field drilling will be advisable for the bottom flanges also. In Fig. 19*i* are shown details of the tension straps and thrust angles for a through truss span; and Fig. 19*j* gives similar details for a deck-girder span. Details of thrust angles to be used at the columns of a deck-girder viaduct are indicated in Fig. 23*c*. The $6'' \times 4'' \times \frac{3}{4}''$ angles noted in this drawing are to be put in place after the longitudinal girder has been erected; and they are ground to fit the diaphragm angles of the column and the end connection angles of the girder. The $\frac{3}{4}''$ plate is used at expansion points only, and is ground to fit the diaphragm angles of the column. In all cases where thrust angles are employed, they must be ground to a tight fit.

The section of the tension flange of the cantilever should be figured at the end of the strap plate; and the bottom flange should be made of the same or an equivalent section, end reinforcing plates being added when required. The flanges should ordinarily consist of two angles, with reinforcing plates on the bottom flanges if needed. Unequal-legged

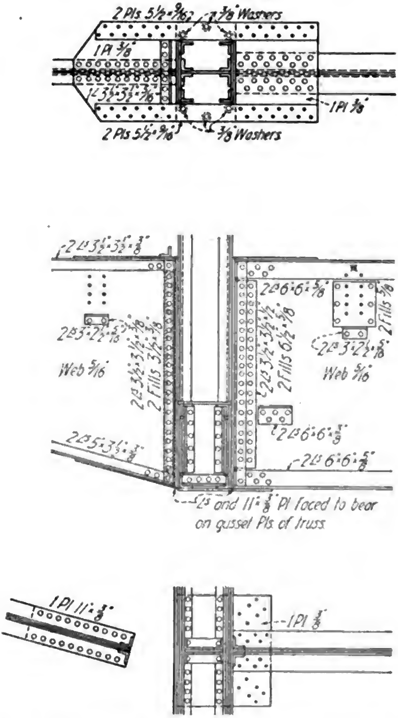


Fig. 19i. Strap Plates and Thrust Angles for Cantilever Beams of Truss Spans.

angles, with the longer legs outstanding, can be used to advantage, as the width of the flange, and, consequently, the allowable unit compressive stress, are increased thereby. As the top and the bottom flanges are nearly always different in detail, the use of the same sections for the two is not so important as in the case of girders with parallel flanges; but usually they should be made the same. For a long cantilever beam a considerable amount of metal is wasted when a single section of angle is used throughout each flange, so that it is frequently advisable to adopt two or more sections, properly spliced to each other. This should rarely be done on any but large cantilevers, as it increases the cost of the shopwork. The bottom flange should generally be braced by brackets to one or more lines of stringers. A detail similar to that shown on the fixed stringer in Fig. 19*h* will serve for this purpose. Such bracing is discussed further in Chapter XX. The end connection angles should conform to the requirements for those of stringers. Details for these are shown in Figs. 19*i* and 19*j*. In cantilever beams it is necessary to mill the bottom flange with the end connection angles in order to secure good bearing. The same is also true of the bottom flanges of the floor-beams where they are of the same depth as the cantilevers. The details at the outer ends should be such that proper connections can be made for the handrails, trolley poles, and lamp posts. While the cantilevers are usually built with solid webs, they are sometimes made of open angle construction, especially where they are short and carry light loads. Fig. 19*e* indicates details for such a cantilever.

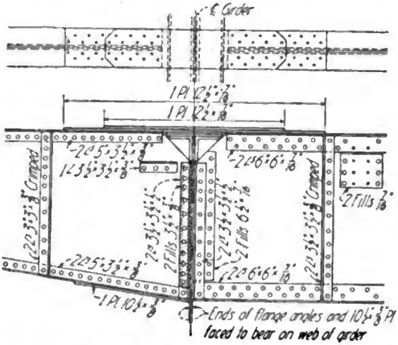


Fig. 19*j*. Strap Plates for Cantilever Beams of Girder Spans.

The handrails for bridges should be designed for the service which they are expected to render. Where there is little likelihood of their being exposed to unusual loads, either transversely or vertically, a light railing can be employed. Such conditions are generally found on small, isolated, highway bridges where the traffic is infrequent and light. However, should the bridge be used by herds of cattle and the like, substantial railings must be provided. Moreover, on city or other bridges where people are likely to collect in large numbers, well designed handrails must be adopted. Not only is it necessary to connect the railing posts themselves firmly to the superstructure, but the railings must be properly attached to the posts.

Various types and heights of handrails are employed. Where they serve as a protection for animals, they should be from 4′ 6″ to 5′ 6″ in height; where used for pedestrian traffic only, along sidewalks, a height of 3′ 6″ or 4′ 0″ is sufficient. Higher railings in such places are objectionable, as they produce the effect of a fence to too great an extent. Sidewalk railings should be so designed that the openings are not over six (6) inches in any dimension, in order to prevent children and small animals from falling through.

For light highway bridges timber railings are sometimes employed. Their construction is fully covered in Chapter LXXVIII. Where steel handrails are used, they may be of either the gas-pipe or latticed type. In both cases they are attached directly to the truss members. In the former, three lines of railing are sufficient. Two (2) -inch pipe is used in the top railing and one and one-half (1½) -inch in the lower lines. The lattice railing is generally made about thirty (30) inches deep with one 2″ × 2″ × ¼″ angle at the top and bottom, connected by 1½″ × 3/16″ bars riveted together where they cross each other.

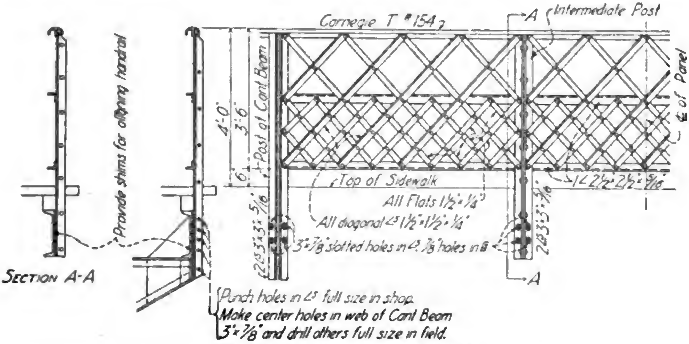

Carnegie T #154
A
Intermediate Post
Provide shims for aligning handrail
4′-0″
3′-6″
6″
Post at Cant Beam
℄ of Panel
Top of Sidewalk
All Flats 1½″×¼″
All diagonal ∠s 1½″×1½″×¼″
1∠ 2½″×2½″×5/16″
2∠s 3″×3″×5/16″
3″×⅞″ slotted holes in ∠s ⅞″ holes in B
Section A-A
A
Punch holes in ∠s full size in shop.
Make center holes in web of Cant Beam 3″×⅞″ and drill others full size in field.

Fig. 19*k*. Lattice-type Handrail.

Gas-pipe railings are seldom employed on city bridges except for the roadways; and in such cases not less than four lines of pipe should be used, the top line being of three (3) -inch diameter and the lower lines of two (2) or two and one-half (2½) -inch pipe. Where posts are required, they are usually of three (3) -inch gas-pipe, cast iron, or structural shapes —preferably small I-beams. The holes in the posts for the railings should have sufficient clearance to permit easy erection, after which the pipes should be wedged tight. Sleeve expansion joints are necessary to take care of temperature changes between the fixed points.

Structural handrails are generally used for city bridges, as they lend themselves better to æsthetic treatment. Many types of railing have been standardized; and their design can be found in any of the catalogues of manufacturers of handrails. The author has employed the railing shown in Fig. 19*k* very extensively. Its appearance is neat, although somewhat severe, and it is easily fabricated by any shop at a moderate cost. Where money is available for ornamentation, more elaborate railings can be designed. Another railing that is inexpensive and at the same time neat and substantial is shown in Fig. 19*l*. With the structural handrails, structural or cast iron posts are employed. For the railing shown in Fig. 19*k* double angles with the wide legs turned out are generally used without braces; for that shown in Fig. 19*l* cast iron or I-beam posts give the most satisfactory details.

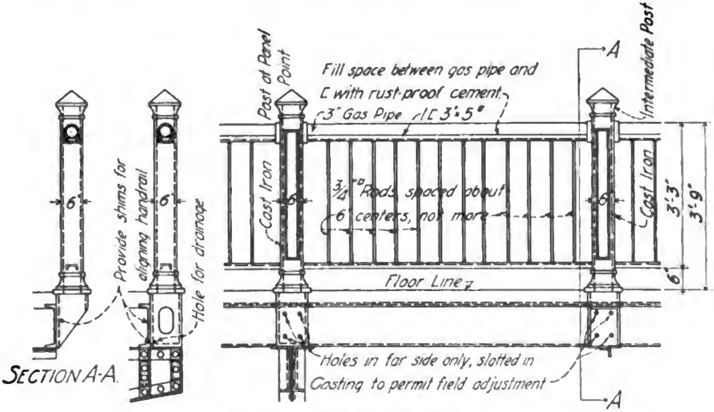

Post at Panel Point
Intermediate Post
A
Fill space between gas pipe and [with rust-proof cement.
3" Gas Pipe
1[3"-5#
Cast Iron
3/4"⌀ Rods spaced about 6" centers, not more
6"
Cast Iron
3'-3"
3'-9"
6"
Floor Line
Holes in far side only, slotted in Casting to permit field adjustment
A
6"
6"
Provide shims for aligning handrail
Hole for drainage
SECTION A-A

Fig. 19*l*. Bar-type Handrail.

In all handrails, proper provision must be made for aligning them in the field both vertically and horizontally, and this should be done in as simple a manner as practicable. Moreover, field drilling should be reduced to the least amount possible or eliminated altogether, as such work is very expensive. It is for this reason that a simple connection should be designed, if possible. Shims are a nuisance in erecting the handrails, but they can seldom be avoided.

With electric cars crossing a structure trolley poles are needed, except where the tracks are between the trusses in through bridges, in which case the trolley wires are connected to the sway frames. Lamp posts are also required in case the structure is lighted, except where brackets can be supported from the trusses in through bridges. Even on through structures it is sometimes advisable to use lamp posts along the outside of the roadways which are cantilevered out from the trusses. Both the lamp posts and the trolley poles may be placed in line with the handrailing or independently along the curbs or some other convenient position. In some cases the trolley poles and lamp posts have been combined into one, but this is not very satisfactory on account of the vibration of the former. Moreover, the separate poles give a much better appearance to the structure, especially if there is a walkway on each side of the roadway, and if the lamp posts can be placed at the curbs and the trolley poles in the handrails, or *vice versa.*

The ordinary gas-pipe trolley pole made up of two or three sections

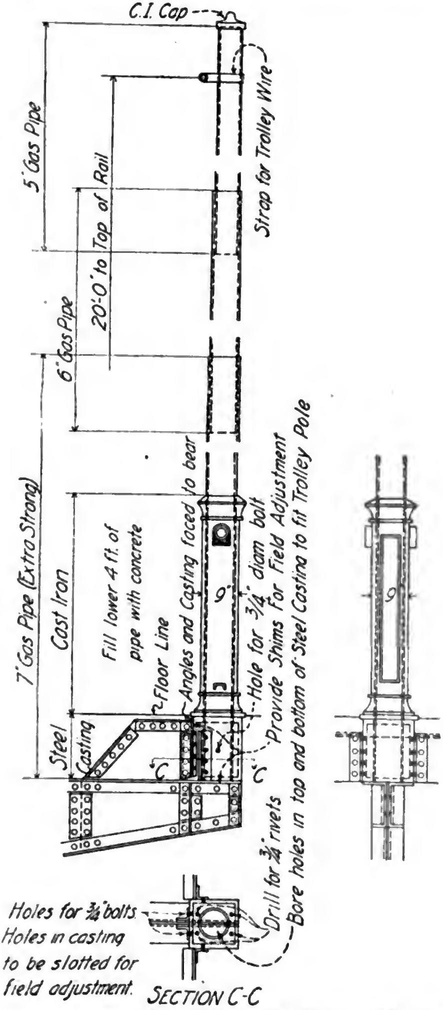

C.I. Cap
Strap for Trolley Wire
5' Gas Pipe
20'-0" to Top of Rail
6' Gas Pipe
7' Gas Pipe (Extra Strong)
Cast Iron
Fill lower 4 ft. of pipe with concrete
Floor Line
Angles and Casting faced to bear
9"
Hole for 3/4" diam bolt
Provide Shims for Field Adjustment
Steel Casting to fit Trolley Pole
Steel Casting
C
C
9
Drill for 3/4" rivets
Bore holes in top and bottom of Steel Casting to fit Trolley Pole
Holes for 3/4" bolts.
Holes in casting to be slotted for field adjustment.
SECTION C-C

Fig. 19*m*. Trolley Pole.

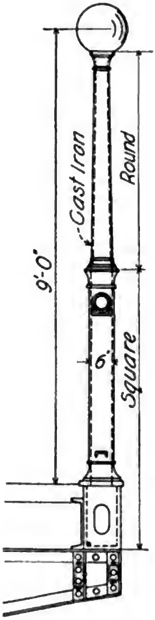


Fig. 19*n*. Lamp Post.

the larger ones swedged to hold the smaller ones at the joints, is generally the best kind to use where the poles are isolated from the handrails. It is supported by a bracket riveted to the steelwork and held in line by a collar attached some distance above it. A modification of this type of pole, designed for use with the handrailing illustrated in Fig. 19*l*, is shown in Fig. 19*m*. A structural trolley pole built up of angles and plates can be used with handrails, although care must be taken to support it properly so that it will not have a tendency to distort. Cast iron should not be employed for trolley poles, unless the stresses therein are very low. They can be used with tension rods on the inside near the outer face. The details for this, however, are not very satisfactory.

Lamp posts are generally made of cast iron of some neat design. Fig. 19*n* shows the lamp post used in connection with the handrailing illustrated in Fig. 19*l*. The lower portion of this post is identical with that of the post at the cantilever bracket shown in the last mentioned figure.

Where a railway crosses a structure on the same level as the roadways, timber screens should be provided between the part of the structure occupied by the tracks and that devoted to the vehicular traffic. When this is not done, horses are more than likely to be frightened by the approaching trains, thus causing serious accidents. Latticed screens are best, as they offer the least resistance to the wind. They should be so arranged, however, that animals can not see through them, and should be high enough so that they cannot look over them

CHAPTER XX

LATERALS AND SWAY BRACING

The laterals of a bridge have to perform three important functions, each of which is vital to the life of the structure.

First. They take care of transverse loads, such as those due to wind or centrifugal forces.

Second. They hold the compression members in line.

Third. They stiffen the structure against the vibrations caused by rapidly-moving live loads; or in other words, they make it rigid.

The need of bracing for actual transverse loads is apparent, and some provision therefor has nearly always been made in bridge construction, at least so far as the main sections of the laterals are concerned. The requirement that all compression members be held in line properly is also evident, although its importance was not fully appreciated in earlier bridge design, and many pony trusses were constructed with inadequately supported top chords. The fact that bridges should be rigid was not appreciated for many years, and as a result many light, vibratory structures were built. It was found, however, that these structures, while perfectly safe at first, soon racked to pieces, and had to be replaced long before their useful life should have ended; and today, no bridge can be considered truly first-class, unless proper rigidity in all its parts has been provided.

The stresses in any given system of laterals from actual or assumed transverse loads can be readily computed. The stress coefficients given in Table 10*h* will be found useful in making such calculations. Wind and traction loads are discussed in Chapter IX, and centrifugal forces in Chapter VIII. For short span bridges, however, it will be found that the sections required for the wind loads are less than those which due regard for rigidity will permit, and in such structures the design of the laterals is largely a matter of judgment. The specifications of Chapter LXXVIII have been so written as to compel, as far as possible, the use of proper sections for such lateral members. For railway bridges, a heavy "vibration load" has been adopted with this purpose in view. All wind loads are treated as moving loads, so as to give greater stresses near the centre of a span than would be found if they were assumed as static over the entire structure. Furthermore, it is specified that, for important highway bridges and all railway bridges, the members of the lateral systems must be capable of resisting compression as well as tension; and this requirement will bar out adjustable rods and light, flimsy angle-members. It is further provided that all detailing must be done in such a manner as to develop the full strength of the members rather than the figured stresses; and a strong, rigid lateral system is thereby assured.

The requirement above mentioned, that the members of the lateral system shall be capable of resisting compression as well as tension, is valuable in that it ensures rigidity, as well as the use of proper sections for strength. When such stiff members are riveted up, the resulting system is certain to be rigid. When riveted members are designed for tension only, the system will be rigid only in case that proper amounts of draw are provided, and that particular care is given to both the shop and the field inspection, so as to make certain that all members will be under ample initial tension. Where the stresses are large enough to ensure fairly large sections, the practice is not so very objectionable; but generally truly stiff members should be used, especially as their adoption rarely calls for much additional metal.

In cheap highway bridges the employment of adjustable rods is permissible; and if they are properly connected to other members, and are kept tightened, they will serve fairly well for such structures.

It should be noted here that with certain types of solid floors the omission of part or all of the lateral bracing is allowable. Where steel trough construction is employed for the floor of a span, all lateral bracing in the plane of the floor should be omitted. Where concrete slabs are used, bracing in the plane of the top flanges of the stringers can usually be left out; and frequently this is permissible for solid timber floors as well. However, under such circumstances it is rarely advisable to omit the lateral bracing between the main girders or trusses, as it will serve to line up the structure during erection. Further, the bridge is likely to stand for some time before the concrete or timber floor is placed; and it may even have to carry traffic. However, there is no objection to using minimum sections throughout for such members.

The centre lines of all members of a lateral system (including the chords of the main trusses when they form the chords of the lateral system) should meet at a common point at each intersection, unless the effect of any resulting eccentricity be considered and duly cared for. The application of the above rule will frequently cause the connection-plates to be excessively large; and if the stresses in the laterals are small in proportion to the stresses in the chords, an eccentric connection is frequently allowable. Where the two leaves of the chord member are well laced together, or have a cover-plate or longitudinal diaphragm, it will be proper to assume that any bending moment coming from the laterals meeting eccentrically at any panel-point affects the chord member at that panel-point only, this moment being divided between the two chord sections meeting at the point in direct proportion to their moments of inertia, and in inverse proportion to their lengths. This action will, of course, put transverse shears on the lacing of the said chord members. In case the two leaves of the chord are joined by tie-plates only, there will be a tendency for any stress applied eccentrically to remain so throughout the entire length of the chord, as the above-mentioned transverse shears are not well provided for; hence with chord members of this type an eccentric intersection of the laterals should usually be avoided.

The make-up of the sections of lateral members depends on the character of the structure, on the stresses they have to carry, on their lengths, and on the connections which are required to suit the main members which they brace. For light highway bridges, adjustable rods are permissible. For all other structures, stiff riveted members are required. Where the members are short and the stresses small, as in stringer bracing, single angles are frequently adopted. For girder spans, and for ordinary truss spans where the laterals can be supported at intermediate points, as from the stringers, two angles placed back to back will usually be found to give a sufficiently large radius of gyration, and to care amply for the stresses which come upon them. Where the radii of gyration required in the two directions are about equal, it is generally best to employ unequal-legged angles, with the longer legs together; although two equal-legged angles will often serve very well. If a greater radius of gyration is necessary in one direction, it can be obtained in three ways. Let us say that the radius of gyration in a vertical direction (about a horizontal axis) is to be the greater. We may then use: two unequal-legged angles having a large difference in the lengths of the two legs, such as seven inches by three and one-half inches (7″ × 3½″) or five inches by three inches (5″ × 3″), with the long legs placed vertically and riveted together; two equal-legged angles, with the horizontal legs riveted together; or two unequal-legged angles, with the shorter legs horizontal and riveted together. The first two methods will make the radius of gyration in a vertical direction about one and one-half times that horizontally, while the third method will make it from two to four times the other. The two angles are, preferably, placed with their backs in contact; but if this arrangement fails to give a large enough radius of gyration, they may be separated by washers. It should be noted that one is not usually free to choose whether the horizontal or the vertical legs will be in contact, as the details of the connections or the proper appearance of the completed structure will generally settle this point.

In case the stresses are larger than the two-angle section will care for without the use of unduly thick angles, a plate may be riveted to them to increase the sectional area.

For sections composed of one or two angles, or of two angles and a plate, the proper reduction in efficiency should be made, as specified in Chapter LXXVIII and explained in Chapter XVI.

When it is impossible to secure a sufficiently great value of the radius of gyration by the use of a two-angle section without undue waste of metal, or when it is preferable to grip both the top and the bottom faces of a chord, an I-strut composed of four angles with a line of lacing is generally preferred. For unimportant members, a channel-shaped strut composed of two angles and a line of lacing is frequently allowable. Ordinarily it will be found best to use unequal-legged angles, with the longer legs turned out so as to get the greatest possible radius of gyration. In case the members are excessively long, it is sometimes necessary to adopt a box strut laced on the four sides, and having at each corner an equal-legged angle with its legs turned inward. The four lines of lacing required make this latter type a rather expensive strut, however.

It will frequently be found possible to effect a saving by supporting a long strut at its mid-point by means of a short strut carried to an adjacent intersection point of the bracing, as the reduced value of the radius of gyration permitted thereby will allow the adoption of a materially smaller section. The possible economy to be effected by this device should always be considered. In detailing the laterals beneath a floor, a saving can frequently be made by supporting them from the stringers.

It is the common practice to use lateral bracing with two systems of cancellation. This custom began in the earlier period of bridge building, when lateral diagonals were nearly always tension members, and the two systems were necessary. With the use of rigid members throughout in the laterals, the double system is not required; but its employment is generally advisable, because of the unsymmetrical appearance of the single system, and, when the main trusses are far apart, because diagonals of a single system would be very long and the necessary radius of gyration hard to secure. For single-track, deck bridges, however, the single-intersection bracing is frequently used; and it effects a material saving, particularly in plate-girder spans. The relative economy of the two types should be considered wherever the appearance of the single system is not objectionable.

Where all the diagonals of a lateral system are tension members, it is necessary, of course, that each diagonal in any panel be able to carry the entire shear in that panel. Where a single system of struts is used, each member must be able to carry the entire shear in either tension or compression; and where a double system of struts is employed, each member should be able to carry one-half of the entire shear in either tension or compression.

A comparatively new system of trussing, known as the K-truss, which is being used in the main trusses of the new Quebec Bridge, has been applied to a limited extent to lateral systems. It can be employed only where the panel length is considerably less than the distance from centre to centre of trusses, under which condition double intersection systems with each diagonal extending over two panels of the floor system have generally been adopted. The two types have the same stresses in the diagonals, and require the same sections, and the diagonals are of equal length; and as the weights of the details are about the same, there is little to choose between the two systems. Messrs. Boller, Hodge, and Baird have used the system to some extent in the end panels of some of their bridges, making the diagonals meet at the centre of the end floor-beam, thereby avoiding the necessity of connecting a lateral member at the shoe, which is always a very troublesome detail to design properly.

It has been the general practice among bridge shops to put "draw" into the diagonals of double-intersection bracing; that is, the members have been shortened slightly, in order to ensure that they shall be under tension when the structure is completed. The American Bridge Company's practice is to use no draw for members under ten (10) feet in length, one-sixteenth ($\frac{1}{16}$) of an inch for members from ten (10) feet to twenty-one (21) feet long, one-eighth ($\frac{1}{8}$) of an inch for members from twenty-one (21) feet to thirty-five feet (35) in length, and three-sixteenths ($\frac{3}{16}$) of an inch for members over thirty-five (35) feet long. This procedure should be followed for diagonals which are in tension only, and for the members of any lateral system which braces compression-chords or columns; but for other stiff bracing no draw need be provided.

The laterals bracing the loaded chords of steam railway and electric railway bridges are usually called upon to transfer traction loads from the stringers to the main girders or trusses, and for this purpose certain additional members are required. The details used for structures of various types are discussed later in the chapter. For stresses due to a combination of wind and traction loads, or of wind, centrifugal, and traction loads, an increase in the unit stresses of thirty (30) per cent over those regularly specified is permissible; but for combined centrifugal and traction loads no increase in the unit stresses is allowable. In case there is a steel trough or concrete floor which can be depended upon to transfer the traction loading to the trusses in an effective manner, no provision for it need be made in the lateral system. For electric railway bridges a solid timber floor is sometimes relied upon to perform this function, but the practice is not to be recommended, as there is usually no effective connection between the flooring and the girders or trusses; and furthermore, the timber floor is likely to become more or less loose and flexible.

The remainder of this chapter will be devoted to a detailed treatment of the laterals and sway bracing of various types of structures.

Railway I Beam Bridges

The only real functions to be performed by the bracing in these bridges are those of holding the compression-flanges of the beams in line, and making the structure rigid. The sections and the connections are generally determined by judgment, as the lightest details consistent with good practice will suffice to carry the figured stresses, excepting only for bridges on very sharp curves. However. the stress in the end diagonal should generally be figured. For bridges on tangent the vibration load should be adopted, and for bridges on curves either the vibration load or the combined wind and centrifugal load—usually the latter. At first thought it would appear that a thirty (30) per cent increase in unit stresses should be allowed for combined wind and centrifugal loads; but as the total load in this case is generally due more to the live load than to the wind, no increase should be made, especially since the vibration load is not assumed to act simultaneously with the centrifugal load. The specifications of Chapter LXXVIII designate in detail the bracing that will be required in spans with various types of floors, and in Figs. 21*t* and 21*u* are shown complete details for two railway I-beam spans with timber deck, the one in Fig. 21*u* having two beams per rail, while that in Fig. 21*t* has one beam per rail. The diagonals are attached to the webs of the beams rather than to the top flanges, in order to avoid interference with the ties. When I-beams of greater depth than twenty-four (24) inches are used, the end sway frames and the intermediate diaphragms should be made of plates and angles instead of channels.

HIGHWAY I-BEAM BRIDGES

The specifications of Chapter LXXVIII cover the bracing of this type of span completely.

RAILWAY DECK-PLATE GIRDER SPANS

The bracing for this type of structure is quite simple, consisting usually of an upper lateral system of diagonal angles attached to the top flanges of the girders, a rigid sway frame at each end of the span, similar intermediate frames spaced not to exceed twelve (12) times the width of the top flange, and, for all spans seventy (70) feet or more in length, a bottom lateral system similar to the upper one. All members of this bracing are rigid struts. The top laterals are unnecessary for bridges having steel trough floors, as was previously explained. Most railway deck plate-girders have open timber decks, however, and for these the top lateral systems are needed. End sway frames are required in practically all cases, as the ends of the girders are rarely concreted solidly into the abutments; and intermediate sway frames are to be used for spans over thirty (30) feet in length.

The top lateral system is to be designed for the same loads as the laterals of a railway I-beam bridge. It consists of rigid diagonal members riveted to the top flange, usually forming a Warren or triangular girder, which may or may not have a cross-strut at each panel-point. For short spans single angles (generally $3\frac{1}{2}'' \times 3\frac{1}{2}''$ or $4'' \times 4''$) will suffice; but for longer spans each member should consist of two angles. For members of the latter type the angles should have unequal legs, the longer ones being vertical and riveted together. They may be separated by washers, if desired, in order to increase the radius of gyration about a vertical axis. The sections must be sufficient for the stresses which they take, the reduced efficiency of the angles being duly considered; and the length must not exceed one hundred and forty (140) times the least radius of gyration. In all cases the capacity of the members will be much less in compression than in tension. There must be enough rivets in each end of each member to transfer the stresses properly, but never less than three (3) rivets for single-angle members or six (6) for two-angle members; and the connection-plates must be attached to the flanges by a sufficient number of rivets to transfer the longitudinal components of the stresses in the diagonals to the said flanges. When there are only two angles in the top flange, it is advisable to put a shim between the connection-plate and the flange angles, of such thickness that the dapped ties will clear the heads of the rivets connecting the diagonals to the plate. In girders having a four-angle top flange, the laterals should fasten to the inside angle of the lower pair, thus avoiding any interference with the ties. For girders on tangent the panel-lengths of the laterals should be such that the top flange will be stayed at points not exceeding twelve (12) times its width, and it is generally economical to make them so short that the gross section of the required tension flange will be ample for the compression flange. Table 21*f* gives, for bottom flanges of the various usual types, the approximate maximum ratios of the unsupported length of the top flange to its width satisfying this condition.

From this table it is evident that where cover plates are used for the flanges, the panels of the top lateral system can be about thirteen (13) feet long (and frequently somewhat longer with little or no increase in the flange section), so that the members are generally best arranged as a simple triangular truss; although for a wide spacing of girders the insertion of a cross-strut at each panel-point may prove more economical. Where no cover plates are used, and where there are two holes out of each angle, the panel-lengths can be about twelve (12) feet long, so that the lateral system can be about the same as in the case of flanges with cover plates. Where no cover plates are employed and when there is only one hole out of each angle, the panel-lengths should be rather short, so that the adoption of a cross-strut at each panel-point of the bracing is nearly always more economical. It should be noted that in any case the top struts of the intermediate sway frames will provide such cross-struts at every second or third panel-point.

The preceding discussion applies only to girders on tangent. For girders on curves, it is specified that the unsupported length of top flange shall not exceed six (6) times its width. This provision is made so that the horizontal bending on the flange may be reduced to such a small amount that it may be neglected. Under these conditions it will nearly always be advisable to adopt the simple triangular truss with intermediate cross-struts at each panel-point for the upper lateral system.

The bottom lateral system, when employed, is to be designed for the vibration load. The panel lengths should be the same as for the top laterals, in order that the detailing of the two systems and of the two flanges may be alike; but no cross-struts will be needed in the lower system. The same make-up of members should be used, but minimum sections and connections can nearly always be adopted throughout.

The end bracing frame consists of a top strut, a bottom strut, and two stiff intersecting diagonals. It should be proportioned to withstand the effect of wind; and if the structure be on a curve, for the effect of the centrifugal force as well. The vibration load is assumed not to affect it. The top strut consists of two angles for a two-angle top flange, and of four angles in the form of an I with a vertical line of lacing for a four-angle top flange. For either type, fillers should be employed between the top flange angles and the connection plates, so that the heads of the rivets in these plates may clear the dapped ties. The bottom strut consists of two angles. Each diagonal consists of either one or two angles, as required by the stresses. The intermediate bracing frames are similar to the end frames in all respects. They have no figured stresses to carry, hence all members should be minimum sections. In Figs. 21*v* and 21*w* are shown typical details for bracing frames for both two-angle and four-angle top flanges.

Double-track, deck, plate-girder bridges usually consist of two single-track spans side by side. For spans over sixty-five (65) feet long a top lateral system should also be put between the two inner girders, as that permits the girder spacing to be made six (6) feet six (6) inches instead of one-tenth of the span, thus saving in ties and avoiding the spreading of tracks. It is satisfactory to put end bracing frames between the inner girders; but intermediate frames should not be used, as they will overstress the outer girder when only one track is loaded.

In Fig. 21*v* are given the complete details of the bracing of a single-track-railway, deck, plate-girder span with the top flange made up of two angles and cover plates; and in Fig. 21*w* are shown typical details of the bracing for a span having the four-angle type of top flange.

RAILWAY, THROUGH, PLATE-GIRDER SPANS

The main lateral bracing of this kind of structure is at or near the plane of the bottom flanges, and is supplied by a rigid system of diagonal angles, or by the floor. When stringers are used, their top flanges must be securely held in line. The top or compression flange of the main girder must also be stayed in an effective manner.

The bottom lateral system is to be designed for the same loads as the laterals of a railway I-beam span. It is usually of double cancellation, the floor-beams serving as the struts of the system. The diagonals generally consist of two angles with the vertical legs riveted together. They may be placed back to back, or spread by washers in order to increase the radius of gyration about a vertical axis. They should be riveted to the stringers where they cross them, in order to reduce the ratio of unsupported length to radius of gyration about a horizontal axis. When four lines of stringers are adopted, the diagonals should be attached to the outer stringers only, since connecting them to the inner stringers would take extra holes out of the flanges thereof at a point close to the centre of the panel, thus wasting metal in the section. For ordinary spans on tangent the l over r requirement, rather than the stresses, will usually determine the section needed. At least three rivets should be employed in the end connection of each angle, and the minimum detail will usually suffice to carry the stresses, unless the structure be on a curve. Fig. 21x shows typical details for the bottom lateral system of an 85′ 9″ railway, through, plate-girder span, designed for the Iowa Central Railway.

The stringer bracing is figured for the vibration load for structures on tangent, and for either the vibration load or combined wind and centrifugal loads for bridges on curves. It is not required when a solid concrete floor which grips the top flanges of the stringers effectively is used. When a timber deck is employed, the bracing is needed. In general, it will be of the same type as that used for I-beam spans, as there will usually be four lines of I-beam stringers per track. Where the panels are longer and two lines of built stringers are employed, the bracing should be of the type later described for through truss spans. When the panel lengths are very short, not exceeding twenty-four times the flange width, a single bracing-point at mid-panel will suffice for I-beams. In this case there should be used diaphragms between the webs of each pair of stringers under one rail, attached to the centre lateral connection plate. Fig. 21x shows a detail of this type of construction.

The bracing of the top flange of a through girder is none too satisfactory, and it should be made as efficient as possible. Where floor-beams are used, the webs thereof should be cut near the ends, and full-depth bracket-plates spliced in, riveting to full depth-stiffeners on the girders. These bracket plates should be as wide as the clearance will permit. Where a steel trough-floor is adopted, similar brackets riveted to the troughs are required. These should be as deep and stiff as practicable. Should the deck consist of long ties resting directly on the bottom flanges or on shelf angles riveted to the girders (which type is forbidden by the specifications of Chapter LXXVIII), the brackets should be riveted to efficient cross-struts. In nearly all cases, it will be economical to make the spacing of these brackets so small that the gross section of the tension flange will suffice for the compression flange. Fig. 21x shows the details of a floor-beam for a through, plate-girder span, with end brackets as before described.

HIGHWAY AND ELECTRIC-RAILWAY, DECK, PLATE-GIRDER SPANS WITHOUT FLOOR-BEAMS AND STRINGERS

This type of structure is rarely found, except for bridges carrying electric-railway traffic only. In this case in all respects the bracing will be similar to that employed in railway spans; but it can usually be made much lighter. For very light highway structures, the diagonals of the laterals and bracing frames may be made of adjustable rods. The lateral system will be designed for the wind loads; and where the structure carries an electric-railway track on a curve, it must be figured for the centrifugal load also.

HIGHWAY AND ELECTRIC-RAILWAY, DECK, PLATE-GIRDER SPANS WITH FLOOR-BEAMS AND STRINGERS

These structures will vary greatly in type, and the laterals required will vary also. The main lateral bracing will generally consist of a double intersection system of rigid diagonals between the main girders, placed just below the stringers, or, preferably, at the elevation of the bottom of the floor-beams. It will generally be found advisable to support these diagonals, at or near the centre of each panel, from the stringers above them. When there is no stringer along the centre line, a good support can be made by running a transverse channel between the two inner stringers at the centre of the panel, and suspending the centre lateral connection plate therefrom by a vertical hanger angle. For light structures, adjustable rods or tension angles may be used for the diagonals.

When a cantilever beam projects from the main girder, as is customary when the width of roadway is considerable, it will generally be necessary to brace the bottom flange of the said cantilever so that the unsupported length thereof shall not exceed twelve (12) times the width, or so as to avoid the use of an excessive section for the said flange. This is generally done by bracing it to one of the stringers by means of a bracket-plate riveted to the said stringer and to a stiffener on the cantilever. Both the stiffener and the bracket plate must extend down to the bottom of the cantilever beam. The stringer is held in position longitudinally by two rigid diagonal struts extending from the centre of the stringer to the points where the cantilevers rivet to the longitudinal girders. These struts should usually be attached to a connecting angle riveted to the web of the stringer, and should lie in a horizontal plane, riveting at the other ends to plates which are fastened by hitch angles to the cantilevers and to the main girders. Usually these diagonals are required in one panel per span only; but the bracket-plates must be employed at every cantilever. Where there is a reinforced concrete slab resting on the top flanges of the stringers and main girders, the diagonal braces above described can usually be omitted. Where a solid timber deck is employed, however, it is not advisable to leave them out.

Diagonal bracing along the top flanges of stringers can ordinarily be omitted when solid floors, either of wood or concrete, are used. Where an open timber deck is employed for street-railway tracks, such bracing will be needed. It should be similar to the top lateral system in railway plate-girders of short span.

When deck girders rest on masonry, there will generally be required some kind of sway bracing at each end, as the bottom of the end cross-girder, if there be one, is almost always some distance above the bottom of the main girder. This can usually be provided by putting a solid-web bracket under each end of the end floor-beam, riveted thereto and to the end stiffeners of the girder. In long-span girders it is generally best to brace the bottom flanges at each intermediate cross-girder as well. Diagonal angles, riveted to the bottom flange of the floor-beam and to stiffeners on the girder, will suffice for this latter purpose.

Occasionally there will be no end floor-beam at one end of a girder span resting directly on the masonry, the stringers being carried by the end floor-beam of the adjacent span. In this case an efficient open-webbed bracing-frame between the two girders of the span should be used, carried up as high as the stringers will permit.

Generally no provision for traction forces will be required in the lateral system when solid floors are employed. Where an open timber deck is used for an electric-railway track, thrust frames to transfer the traction loads to the main girders or columns should be provided. They should consist of horizontal trusses, usually placed at the plane of the bottom flanges of the stringers, with a floor-beam forming one chord of the truss.

The details of the bracing of steel columns and towers will be taken up in Chapter XXIII under the discussion of trestles and approaches.

HIGHWAY AND ELECTRIC-RAILWAY, THROUGH, PLATE-GIRDER SPANS

The bracing of spans of this type will follow in all essential respects that for railway through spans, except that the sections may be lighter, the use of tension angles or adjustable rods for the diagonals of the lateral system being permissible in very light structures. Under no circumstances, however, should the top flanges of the main girders be inefficiently braced.

RAILWAY, THROUGH, TRUSS SPANS

The lateral bracing of this type of structure usually consists of the following:

1. The stringer bracing.
2. The bottom lateral system.
3. The top lateral system.
4. The intermediate vertical sway-bracing.
5. The portal bracing.

The stringer bracing, when only two stringers per track are employed, consists of a system of diagonal angles riveted to the top flanges of the stringers, with a bracing frame at mid-panel between stringers of the same track when the length exceeds thirty (30) feet. In case the stringers rest in expansion pockets, there should be a cross-frame near the ends between the stringers of the same track. The proportioning will follow in every respect that for the top lateral system and the sway frames of deck plate-girder spans. The bracing will generally be arranged as a triangular truss with intermediate cross-struts, as it will usually be necessary to brace the top flanges about every six or seven feet. When two holes are taken out of each angle of the tension flange, the flanges will need stiffening about every twelve feet (for 6″ outstanding legs); and in this case, for structures on tangent, the intermediate cross-struts can be omitted. In Fig. *22fff* will be found details of stringer bracing.

When four I-beam stringers per track are adopted, the bracing should be similar to that used for I-beam spans. If very short panels are employed, the details given for through-girder spans are generally satisfactory. When four built stringers per track are used, and a point of transverse support for the top flange at the centre of the panel only is sufficient, the detail shown in Fig. 21*x* is best. When several points of support are needed, a single intersection system of single diagonal angles should be used between the top flanges of the two inner stringers, and a transverse strut running over to the outer stringer should be employed at each panel-point of the bracing.

The bottom lateral system is in the plane of the bottom chords. For bridges on tangent, it must be figured to carry either the vibration load or the wind load. For bridges on curves, it must care for either the vibration load or the combined wind and centrifugal loads. The diagonals will also have to be figured for the traction load, as explained later. The bracing system is nearly always of double cancellation, with the floor-beams forming the struts. Usually each diagonal extends over one panel of the truss; but when the panel-length is much less than the distance between the centres of trusses, a decided economy can be effected by making each diagonal extend over two panels. The same result is sometimes obtained by employing the K-type of trussing. For ordinary spans the diagonals usually consist of two angles with the vertical legs upstanding and riveted together; and they are generally placed in the plane of the bottom of the floor-beams, with the connection plates riveted thereto. In riveted trusses with bottom chords consisting of two channels, the connection plates rivet to the bottom surface of the said chords. In riveted trusses with four-angle bottom chords, the centre line of the chord is usually at the bottom of the floor-beam, and the lateral connection plate fastens to the chord by connection angles. In pin-connected trusses with eye-bar bottom chords, the vertical posts are extended down to the bottom of the floor-beams, and the lateral connection plates are riveted thereto by means of connection angles. In riveted trusses with bottom chords consisting of two channels—either rolled or built—and in all pin-connected trusses of short span the longitudinal component of the stress from the laterals is applied below the centre of the chord. In riveted truss bridges, the moment caused by this eccentricity is divided among all the truss members at the point; while in pin-connected truss bridges the post alone takes this bending. These bending effects should always be properly considered. In large bridges, it is generally advisable to avoid them by employing bottom lateral diagonals of four angles in the form of an I with a vertical line of lacing. For riveted trusses, the diagonals should be made as deep as the chords. The bottom connection is as before, while the top connection plate rivets to connection angles on the floor-beam web and on the chord gusset plate. For pin-connected trusses, the diagonals should, preferably, be deep enough to bring the centre lines of the said diagonals and of the chords into the same plane. The bottom connection is similar to that used for the two-angle section. The top connection plate is attached by connection angles to the floor-beam web and to the post.

The rivets in the connections of the laterals must be able to develop the section thereof. The rivets fastening any lateral connection plate to the truss must be able to transfer thereto the longitudinal component of the stresses in the lateral diagonal or diagonals which it connects, and those connecting the said plate to the floor-beam must be able to care for the transverse components of the diagonal stresses.

The connection plate on the end floor-beam will be found very troublesome to design, owing to the reduction in its strength caused by cutting out a portion to clear the shoe. For a double-track bridge it will be frequently possible to employ the detail used by Boller and Hodge on the Municipal Bridge at St. Louis, in which for the end panel the diagonals were made to intersect at the centre of the end floor-beam, as has been previously mentioned. This detail could not be used for a single-track structure unless the panels were very short, as the angle between the diagonals and the chords would be too small.

It is desirable that the members of the bottom lateral system intersect on the centre line of truss, especially as this result can usually be accomplished with little waste in the connection plates. For riveted trusses with laced bottom chords, in which the stresses from lateral loading are only a small proportion of the total, an eccentric connection will usually do no harm; but its effect should generally be figured, as explained previously.

The bottom laterals in most cases should be connected to every stringer they cross. With four-angle laterals the bottom of the stringers should be at the same elevation as the top of the laterals, so that the top angles of the latter can rivet directly to the bottom flanges of the former; but with the two-angle type of lateral the use of clip angles will be necessary. Fig. 22*fff* illustrates a detail of this type. It was previously mentioned that the laterals should be utilized to care for traction stresses. In single track bridges with two lines of stringers all that is necessary is a transverse strut at each point where the laterals cross the stringers, which strut with the lateral diagonals and the floor-beam forms a complete truss for transferring the traction loads from the two stringers to the trusses. For single-track bridges with four lines of stringers, the lateral diagonals should be riveted to the two outer lines only, for the reason given when discussing railway through-plate-girder spans. The transverse strut is to be used in the same manner as for spans with two lines of stringers. In multiple-track bridges with two lines of stringers per track (as is nearly always the case), the diagonals should be riveted to the stringers at every intersection. The traction loads should be cared for by placing one thrust frame in each panel. Each thrust frame is best made by placing a transverse strut between the points where the lateral diagonals intersect the inner stringers, and then running diagonal struts between the points where the lateral diagonals intersect the outer stringers and the points where the inner stringers intersect the cross girder, and also between the inner stringers. There is thus formed a complete truss having the floor-beam and the transverse strut for the chords, the lateral diagonals for the end posts, and the stringers and the diagonal struts for the web members. The members of this truss should be figured for the specified traction load, acting on one track in either direction, or on both tracks simultaneously, in either the same or opposite directions. Where double-plane bottom laterals are employed, the thrust frame should be single plane, and only the upper pair of angles in the laterals should then be relied upon for traction effects.

Another feature of the action of the thrust angles or thrust frames should be noted. When the bottom chord elongates, the stringers tend to remain of constant length, and hence they put very severe bending stresses in the floor-beams. These are relieved to some extent by the use of wide-legged connection angles for the stringers, but the effect is still large. The thrust angles in every panel tend to cause the stringers to move with the chords, and, therefore, relieve the floor-beams decidedly.

In case a bridge has a trough floor system carried by the bottom chords, all bottom laterals and traction frames are, of course, omitted.

The top lateral system is in the plane of the top chord. It should be proportioned to carry either the vibration load or the wind load. It usually consists of a double intersection system of rigid diagonals, with a cross strut at each panel point of the truss, which strut generally acts also as the top strut of the sway frame or portal. Usually the diagonals extend over one panel only; but where the panel length is much less than the distance from centre to centre of trusses, a considerable economy can be effected by making each one extend over two panels. The same effect can also be obtained by using the K-system of trussing. All members of the bracing should be of the same depth as the top chord, so as to grip both faces of the latter effectively. The diagonals usually consist of four angles in the form of an I, with a single line of lacing. Unequal-legged angles, with the longer leg outstanding, are generally to be employed. It will be found that for all ordinary spans, four angles 3½″ × 3″ × ⅜″ can be used for these struts. For short members, two such angles with a vertical line of lacing will frequently suffice.

The top lateral diagonals are often made to intersect on the outer gauge line of the cover plate in order to keep down the sizes of the connection plates. Since the lateral forces are comparatively small, this will do no harm in bridges of ordinary span, as the top chords are always effectively laced. For long span bridges, the effect of the eccentricity thus involved should be considered and either provided for by using extra metal in the section, when necessary, or else avoided by adopting a truly centric intersection of axes.

The connections of the diagonals are usually to be proportioned for the development of the capacities of the members, rather than for the figured stresses. The connections of the plates to the trusses and to the transverse struts will generally be a question of proper detailing—not one of required strength. The longitudinal and transverse components of the stresses in the diagonals which connect to each plate must, of course, be properly taken care of.

The specifications of Chapter LXXVIII give the requirements which the lacing and the tie plates must meet. Usually the lacing is of the single-intersection type; but in deep members double-intersection bar lacing or latticing, or even angle lacing, is sometimes necessary.

Vertical sway bracing is usually placed at every main panel point of the top chord except at the end ones, where the portal braces are located. It is provided primarily to stiffen the structure, and is usually made throughout of the minimum members which will meet the *l* over *r* requirements. Sometimes it is figured to transfer to the bottom laterals half of a top chord panel wind load. Sway frames increase the bending in posts from floor-beam deflection, and their omission on this basis has sometimes been proposed; but the author does not consider this advisable.

The vertical sway bracing is ordinarily of the single plane type for single-track bridges. The type outlined in Fig. 20*a* is generally used with straight top chords, and that in Fig. 20*b* with polygonal top chords. The top strut is of the same form as that adopted for the upper lateral diagonals; and in most cases it can be made of the same section. The other members are often of two angles each. There should always be either a vertical diaphragm or a pair of batten plates on the post at the point where either the bottom strut of the bracing frame or the corner bracket connects to it. The weak point of this sway bracing is this connection, as the transverse loads are delivered to the comparatively thin webs of the posts, rather than directly to the lines of lacing. It is possible to put in transverse diaphragms with horizontal webs connecting to tie plates so as to strengthen this point; but for ordinary spans it is unnecessary to do so.

For double-track bridges, and for very long and heavy single-track bridges, the use of the double-plane sway bracing is best. A type similar to that outlined in Fig. 20*a* is generally employed for bridges with straight top chords, and one similar to that in Fig. 20*b* for bridges with polygonal top chords. The top strut is either a box-strut with an angle at each corner and lacing on all four sides, or else an I-strut as in the single plane type of sway bracing. In this case the strut should usually be supported laterally at mid-length from the centre connection plate of the lateral diagonals, as the saving in the section of the strut will ordinarily give more than sufficient metal to build the supporting strut. The other members of the sway frame consist of two or four angles with a horizontal line of lacing. The connection plates to the posts should extend across the faces thereof as tie-plates. This connection to the posts has none of the weakness of that of the single-plane type of sway bracing.

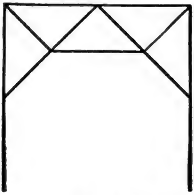


Fig. 20*a*. Outline of Sway Bracing Frame for Through Bridges with Parallel Chords.

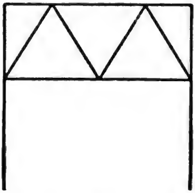


Fig. 20*b*. Outline of Sway Bracing Frame for Through Bridges with Polygonal Top Chords.

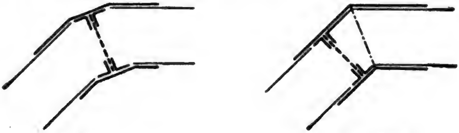


Fig. 20*c*. **Fig. 20*d*.**

Arrangements of Top Struts of Portals.

The portals are practically always placed in the plane of the end posts. They should invariably be of double-plane, thus gripping both faces of the end posts very effectively. The portal is proportioned to resist the effects of the wind pressure. It is generally assumed that the entire wind load on the top chord is carried to the portals, and then transferred down through these and the end-posts to the shoes. The assumption is usually made that the end posts are fixed in direction at the pin, and are held in line at the top and at the bottom struts of the portal bracing, thus putting a point of contra-flexure between the pin and the bottom portal strut. The position of this point can be taken from the upper curve of Fig. 16*d*. It will be noticed that it is always a little over halfway up from the pin. After the point of contra-flexure is known, the stresses in the end posts and portals can be determined by the principles of statics, as explained in various standard works on bridges—notably in "Modern Framed Structures," Parts I and II.

The top strut of the portal is best made of the form shown in Fig. 22*eee*. A fairly efficient top strut can be made of a four-angle I-strut with a single line of lacing, the strut being placed on the bevel between the two members, as shown in Fig. 20*c*, or on the end post as shown in Fig. 20*d*. The other members are generally four-angle I-struts, although two-angle channel-struts are occasionally employed. In proportioning the connections, it will be found that many of them are governed by the requirements of good detailing rather than those of stresses. Care should be taken to ensure that all components of the stresses in the various members are duly provided for.

In skew bridges, the detailing of the portal is an especially difficult matter; and particular attention is necessary in order to make the connections to the end-posts even reasonably efficient. An article in the *Engineering News* of February 11, 1909, page 152, and one in the *Engineering Record* of February 17, 1912, page 196, show methods to be followed in drawing the details of skew portals.

RAILWAY, DECK, TRUSS SPANS

There are two principal types of these structures. In one, there is no steel floor-system, the ties resting directly on the top chords. This type is used for short spans only. For long spans, stringers and floor-beams are required.

In both of these types of structure, the following lateral bracing is needed:

1. The top lateral bracing.
2. The bottom lateral bracing.
3. The intermediate sway frames.
4. The end sway frames.

When stringers and floor-beams are used, there is also required:

5. Stringer bracing.

The bracing for the first mentioned kind of structure will first be discussed.

The top laterals can be of either single or double cancellation, the former usually being the more economical. In this case there will generally be two lateral panels per truss panel. There will be a cross-strut at each panel point, furnished by the top strut of the sway frame. This upper lateral system is proportioned for the same loads as the bottom lateral system of a through span. The diagonals usually consist of four angles in the form of an I, with a vertical line of lacing. This lacing is generally of angles, as the top chords are very deep in trusses of this type. It is ordinarily a simple matter to make the diagonals intersect on the centre line of the truss, and this rule should generally be followed, if practicable; but intersectiong them near the edge of the chord is not very objectionable. The connection of each lateral plate to the chord must be sufficiently strong to care for the longitudinal components of the two diagonals meeting upon it.

The bottom lateral system of a deck-bridge is not very important. It should usually be made of the same type and panel lengths as the top lateral system, with minimum sections and connections throughout. It should be proportioned for the same loads as the top lateral system of a through span. The members should be of the same depth as the bottom chord; and as this member is generally shallow, bar lacing will ordinarily suffice.

Intermediate sway frames are used at every panel point. In order to ensure that an effective frame will be adopted, it is specified in Chapter LXXVIII that it shall be strong enough to transfer one-half of the live load concentration on one truss over to the other truss. The top strut should be as deep as the top chord, and it is generally made of the same form as the top lateral diagonals. The bottom strut should be of the same depth as the bottom chord, being usually of the same make-up as the lower lateral diagonals. The diagonals of the sway frames are of double cancellation, each member being composed of two unequal-legged angles with the short legs vertical and riveted together. Care must be taken to see that the vertical component of the stress in each diagonal is properly transferred to the truss, and the horizontal component thereof to the lateral connection plate.

The end sway-frame must be proportioned to withstand the entire transverse loading carried to one end of the truss due to the wind load if the structure is on a tangent, or to the combined wind and centrifugal loads if it is on a curve. The top and bottom struts are generally of the same section as those of the intermediate frames. It is preferable that the diagonals be double-plane and of the same width as the end posts of the truss, in order to grip their faces more effectively. The diagonals should consist of I-struts of four angles with a single line of lacing. The horizontal load at the top is delivered to the diagonals by the upper laterals and the top strut, and is transferred at the bottom to the end post, or, preferably, directly to the shoes. The vertical component of the diagonal in each case is delivered to the end post. Enough rivets must be provided to care for these stresses.

A bridge of this kind might occasionally be built with a steel trough floor. In that case all top laterals would be omitted, but efficient connections for the upper ends of the sway bracing diagonals would have to be provided.

The lateral bracing of deck spans having floor-beams and stringers will next be discussed.

The top laterals are usually of double cancellation, each diagonal extending over one truss panel, with the floor beams serving as the struts. This lateral system is proportioned for the same loads as the bottom lateral system of a through span. As a general rule, in spans of this type, the tops of the floor-beams lie just below the bottom of the top chord, and the tops of the stringers are a little below the tops of the floor-beams. The lateral connection plates are placed between the bottom of the top chord and the top of the floor-beam. The best arrangement, so far as the laterals are concerned, is to make each diagonal of two unequal-legged angles with the long legs upstanding and riveted together. These angles continue across and rivet to the top of each stringer, the use of thick filler plates between the stringers and laterals being necessary. This style of laterals requires the use of longitudinal wooden shims on top of the stringers, so that the ties may clear the laterals. If this detail be considered objectionable, the vertical legs of the diagonals can be turned down, and the angles cut where they cross each stringer and spliced by connection plates which rivet to the stringer flanges. This arrangement cuts each lateral up into several short pieces and is, therefore, not as good as the type first described.

The stringer bracing is usually supplied by adding a few short members to the top lateral system. In single track bridges with two lines of stringers (which is the type nearly always employed), it is generally sufficient to place transverse struts between the stringers at the points where the diagonals intersect them, and a diagonal running from one end of each of these struts to the point where the other line of stringers meets the floor-beams. If the unsupported length of the centre portion of the top flange is too great, it will be necessary to run a transverse strut from the central lateral connection plate over to each stringer. In stringers over thirty feet long, a cross-frame at mid-panel should be used, and this will supply the centre transverse strut. In single-track bridges with four lines of stringers, the transverse struts should be placed at the points where the laterals intersect the outer stringers; and in this case the centre transverse strut will nearly always be required. If the unsupported length of the end portion of the top flange of the stringer be too long, it can be braced by using, instead of the single diagonal above mentioned, two panels of single intersection bracing of the Warren or triangular type with a cross strut at its mid-point riveting to all of the stringers. In double-track bridges, it will be necessary to add a sufficient number of diagonals and transverse struts to brace the top flanges properly. The laterals should be utilized for this bracing as far as possible. In case stringers rest in expansion joints, there should be an end sway frame between the stringers of each track.

The top laterals must also care for traction loads. In single-track bridges the members already specified for stringer bracing will serve to carry traction loads as well. In double-track bridges the stringer bracing can be arranged in such a manner that the addition of two small members per panel will provide for the traction loads.

The bottom laterals are of small importance. They usually consist of a double intersection system of diagonals, the bottom struts of the sway frames forming the cross-struts of the system. The diagonals generally consist of four-angle I-struts with a vertical line of bar lacing, and are of the same depth as the bottom chord.

The sway frames are about the same as for the other type of deck span, except that the top strut is replaced by a floor-beam. It is frequently best, however, to make the diagonals of the intermediate sway frames the same as those described for the end sway frames on account of the long members involved. They are proportioned for the same loads as those for the other kind of span.

HIGHWAY AND ELECTRIC-RAILWAY, THROUGH, TRUSS SPANS

The laterals for these spans will not vary essentially from those used for railway spans except in light structures. The loads for which they are to be proportioned will usually be less, and there is no vibration load to be considered. The effect of centrifugal force, when there are electric railway tracks on curves, should be considered; and traction from electric-railways should also be provided for, unless there is a concrete slab extending practically the full width between trusses. Stringer bracing should be omitted when a concrete floor slab is used, and generally with a solid timber floor. The bottom lateral diagonals will frequently lie at a considerable distance below the stringers, and in this case they should be supported at mid-panel in the manner explained for highway plate-girder spans. Cantilever brackets outside of the trusses can be braced in a similar manner to that explained for highway plate-girder spans; but in this case the diagonal braces should connect to the bottom flanges of the cantilevers at the line of stringers from which they are braced, and their other ends should rivet to a bent plate on the bottom of the lower chord at the centre of the panel. There will also be required a diaphragm or bracket-plate attaching the stringer to the connection plate of the diagonal. This bracing for the cantilevers will generally be used in two panels per truss; and where a solid concrete floor is employed, it may be omitted altogether.

For light highway bridges it will be permissible to employ angles in tension, or even adjustable rods, for the diagonals in bracing of double cancellation. When tension angles are adopted, the draw mentioned previously should be provided. The use of pony truss spans should be discouraged for even light bridges, but sometimes it is necessary to employ them. In that case the top chords must be braced to the floor-beams as rigidly as possible.

HIGHWAY AND ELECTRIC-RAILWAY, DECK, TRUSS SPANS

The bracing of structures of this type will in essential particulars follow that for railway, deck, truss spans. The special points which will arise will be very similar to those described for highway through, truss, spans; and they should be treated in a similar manner.

CHAPTER XXI

PLATE-GIRDER AND ROLLED I-BEAM BRIDGES

ALTHOUGH plate-girders are of necessity as unscientific structures as a bridge specialist ever has to design, they are without doubt the most satisfactory type of construction possible for short spans. Their superiority over articulated trusses is due to the following reasons:

First. Owing to their compactness they better resist shock and check vibration.

Second. They have fewer critical points where overstress is likely to exist because of faults of either designing or workmanship.

Third. A number of loose rivets lying close together will do far less harm in a plate-girder than in an open-webbed one.

Fourth. The cost of manufacture per pound of metal is a little less.

Fifth. Owing to the steady demand for plate-girder structures and the comparative simplicity of the sections of metal used in their manufacture, it is easy to obtain quickly the materials required; and the work on the metal is of a simple character. For these reasons plate-girder spans can generally be purchased with less delay than open-webbed girders.

Sixth. The cost per pound for erection is decidedly less, excepting where the conditions are unusual.

Seventh. They can be overstressed without danger much higher than open-webbed girders.

Eighth. They are less liable to injury by accident than articulated trusses.

Ninth. They are more easily painted, and are more accessible to examination for rust.

Tenth. The cost of maintenance is less, owing to the absence of small parts and details that might work loose under traffic.

One rarely hears of the failure of a plate-girder span. while the collapsing of open-webbed girders (especially old, pin-connected ones) is far from uncommon. There has lately come to the author's notice an old, wrought-iron, plate-girder bridge of eighteen and a half feet span, in which, when impact was included, the actual locomotive loads stressed the extreme fibres of the bottom flanges as high as forty-two thousand (42,000) pounds per square inch. How the structure continued for years to stand up under the constant traffic on one of the great trunk lines is a puzzle for bridge experts; because the elastic limit of the metal must have been less than thirty thousand (30,000) pounds per square inch. Stressing an old-fashioned, iron, pin-connected-truss span one-half as much as that plate-girder bridge was stressed would probably have caused its downfall long ago.

The ordinary limit of length of plate-girder spans is about one hundred (100) feet, but that limit has often been surpassed by twenty-five (25) or thirty (30) per cent for simple spans and by much more for swing spans. Usually it is the difficulty in shipping very long plate-girders from bridge shop to site that determines the superior limit of such spans. The loading of long girders on cars for shipment is quite an art, and it should be entrusted only to men experienced in such loadings; for, otherwise, the metal is liable to be injured in transit or the cars to break down, or some other trouble is likely to happen before they reach their destination. Some engineers believe that the liability to injury of long plate-girders in shop, transit, and field should limit their length to one hundred (100) feet; but the author is not of this opinion, for he thinks that by taking proper precautions the danger can be pretty nearly eliminated. About as long a plate-girder as has ever been shipped in one piece was one of one hundred and thirty-two (132) feet. It required four flat cars to transport it. Longer plate-girder spans than this have been built, notably tubular bridges and swing spans, but they were shipped in parts and assembled at site. This expedient for simple spans is really permissible only in case of bridges to be sent to foreign countries, and it is to be avoided if possible even then, because it is sometimes difficult to obtain a satisfactory job of field-riveting when making the splices, although the use of pneumatic riveters tends to reduce materially the force of this objection. The specifications of Chapter LXXVIII provide an excess of strength for the field splicing of plate-girders, simply as a matter of precaution for the avoidance of the possible ill-effects of defective field riveting.

As far as economics are concerned, it may be stated that, if deck plate-girders are feasible for any opening, they are more economical than truss spans up to a length that is prohibitory for shipment. As the depth of a very long plate-girder is generally from one-tenth ($^1/_{10}$) to one-twelfth ($^1/_{12}$) of the span, the requirements of underneath clearance often bar out deck plate-girders and necessitate either half-through plate-girders or through trusses. The rule given in Chapter LXXVIII, that in single-track deck-bridges the perpendicular distance between the webs of any pair of plate-girders must not be less than one-tenth ($^1/_{10}$) of the span, limits the span-length for such structures to about one hundred (100) feet, because a greater length than ten (10) feet between centres of bearings of ties would involve the adoption of heavier timber than is easily and economically procurable. But for double-track structures the use of four lines of girders spaced equidistant permits the bracing together of two pairs of girders by vertical end frames, and the running of both the upper lateral system and the lower lateral system from outer girder to outer girder. As wooden ties of ordinary dimensions can be used under such circumstances, the limiting length of the girders will be determined only by the ability of the railroads to transport them. It is not likely that girders exceeding one hundred and twenty (120) feet in length can be carried on cars advantageously; and, moreover, a riveted truss span of that length, having four or five panels, makes a very satisfactory structure. Some bridge engineers are not governed by the rule that the perpendicular distance between central planes of plate-girders in single-track deck-bridges must not be less than one-tenth ($^1/_{10}$) of the span; hence for them the superior limit for the spans of such structures might readily be much more than one hundred (100) feet—in fact, there are on record long-span bridges of this kind in which the width is less than one-fourteenth ($^1/_{14}$) of the span. While this ratio is certainly extreme, it must be confessed that the author would not hesitate to violate his own rule somewhat to meet unusual conditions, because it was made for the sole purpose of checking lateral vibration under heavy loads passing at very high speed; therefore, if these conditions could not exist, there would be good reason for modifying this item of his specifications.

In the case of half-through, plate-girder spans or deck plate-girder spans with steel stringers and floor-beams, it will be found that these are more expensive by several dollars per lineal foot than the corresponding riveted through- or deck-truss spans for a length of one hundred (100) feet, and that for greater lengths the difference of cost is still more. As it hardly seems advisable to build riveted truss spans shorter than one hundred (100) feet, it is well to adopt this length as the superior limit for half-through plate-girders and deck plate-girders with steel floor systems.

Considerable expense in erection can often be saved by riveting up entire railroad, deck, plate-girder spans in the shop, shipping them to the site, running them into horizontal position, lifting them off the cars by gallows frames, running the cars from beneath them, and lowering them into place. This can be done quite readily with spans up to sixty (60) feet long or even longer. Provided that the assembled girders do not require too much width for the cars that are to transport them, this arrangement will work very well.

Rolled I-beam spans are preferable to plate-girder spans up to the limit of economy and even beyond it, provided that the deflection under load does not become too great. For ordinary I-beams the longest railway spans of this type are about twenty (20) feet when four lines of stringers per track are used, but by employing the thirty (30) inch special sections of the Bethlehem Steel Company the limit can be increased to about thirty feet for fairly heavy engine loads. By using six lines of beams per track the limit can be increased to about thirty-five (35) feet, for which span the depth is only one-fourteenth ($^1/_{14}$) of the length—a ratio common enough in England, but objected to by most American engineers because of the great deflection that it permits. The superiority of rolled I-beam spans to plate-girder spans lies in their simplicity of detail, their greater compactness, and their lower pound price.

Some years ago there were designed for a transcontinental line a number of plate-lattice girder spans. Their *raison d'être* was supposed to be primarily their ability to pass water through them when submerged, but secondarily, economy. The designer claimed that they effected a saving of metal amounting to about fifteen hundred (1500) pounds for an eighty (80) foot, single-track span, and that the pound price for their manufacture was no greater than that for ordinary plate-girder work. The author once used plate-lattice girders for the cross-girders of the Union Loop Elevated Railroad of Chicago, but his object was simply to evade a troublesome clause in the city ordinance. The webs of these cross-girders were solid near mid-span and at the ends, and were open near the quarter points, while those of the railroad girders previously mentioned were solid at the ends and open over more than the middle half of the total length. As far as the author's experience goes, it takes just as much metal to build the webs open, and the pound price for the finished metal is a trifle greater than it is for ordinary plate-girder construction. The fact that this same railroad, when drawing up a set of standard plans a few years later, discarded the plate-lattice girders is a pretty sure indication that the advantages claimed for them were more imaginary than real. It is true, of course, that in case of submergence they would pass a certain amount of water through their webs; but it is seldom that a railroad company will build a bridge of any kind so close to the high-water mark as to run any risk of its being submerged.

As the calculation of stresses in beams is explained clearly in a number of standard text-books, it will be assumed that the reader is familiar with the subject in general, and this chapter will discuss it merely from the standpoint of its direct application to the design of girders. The use of equivalent uniform loads is treated fully in Chapter X.

A plate-girder is a beam composed essentially of a wide, thin plate, called the web, along each edge of which there is riveted a flange that may consist of various structural shapes. The loads which the girder carries produce at any section both shear and bending moment. The web is usually considered to care for the shear, while most or all of the bending moment is assumed to be taken by the flanges. In additicn to the flanges and web there are many other important parts, each of which has its own function to perform. Some of these, such as the end-stiffeners or end-connection angles, are found in nearly every plate-girder; while others, such as web-splices and flange-splices, are used only when the conditions of any particular case call for them.

The most important calculations are those for the maximum moment and the maximum end shear. For rolled I-beams and short-span plate-girders these are usually the only ones required; but for a plate-girder of longer span, it is generally necessary to know also the laws of the variation of moment and shear throughout its length, so that the values of these functions at any given section can be determined. These laws will depend upon the kind of loading (*i. e.*, live or dead), on the manner in which it is applied to the girder (*i. e.*, uniformly distributed or concentrated), and on the end conditions of the girder. The girder simply supported at both ends will first be discussed at some length after which several other forms will be considered briefly.

For a girder carrying a uniform load the maximum shear on the web is simply the load between the point of support and the centre. If there is no end overhang, this quantity is also the maximum reaction on the support; but if there is such an overhang, the load on it must be considered in finding the said reaction. The dead-load shear at any point is equal to the dead load between the section in question and the centre of the girder, varying uniformly from zero at that point to a maximum at the end. For advancing live load covering a portion kl of a span of length l, the shear at the head of the load is k^2 times the end shear. In Fig. 21*a* are plotted values for maximum shears from uniform loads at various sections of a girder, for varying ratios of live and dead loads. These are given in ratios of the end shear. (The k on this diagram is not the same as that just used above, being measured from the other end of the girder.) The moment diagram will be a parabola, with the maximum ordinate at the centre.

In a railway girder without panels, such as an ordinary deck girder or a through girder with a trough floor system, the dead load is uniform, and the shears and moments caused by it can be figured in the manner just explained. The maximum live-load shear on the web can be taken from the curves of Fig. 6*c*. If there is no end overhang, this will also be the maximum reaction on the support; but if a girder of length l have an end overhang a, the maximum reaction on the support will be that given in the curves for a span of length $l + a$, multiplied by the ratio $\frac{l + a}{l}$. The shear at a point distant kl from one support of a span of length l, due to a live load on the portion kl, may be taken as k times the end shear of a span of length kl. There is no exact law for the variation of live-load shears throughout a span, as it differs for different span lengths; but the curve of Fig. 21*b* will be found to be sufficiently accurate for designing purposes. It gives values of the ratio of the total shear at any section to the total end shear on the web. The maximum live-load moment at the centre is to be figured by the use of the equivalent uniform live load; and the moment diagram can be assumed to be a parabola, although the actual moment curve will usually be a trifle higher at some points on account of the wheel concentrations. This error is easily taken care of, however, as will be explained when the figuring of cover-plate lengths is discussed.

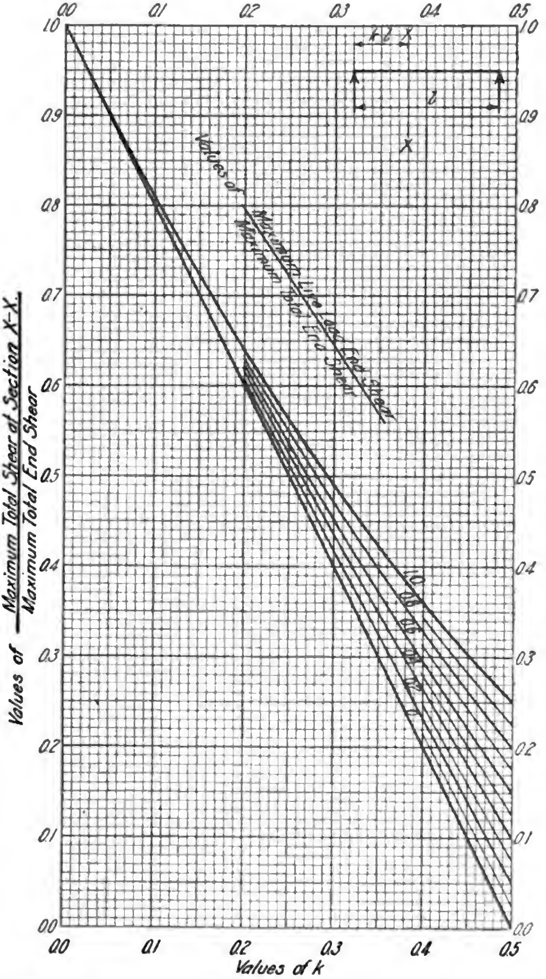


FIG. 21*a*. Total Shears throughout Plate-girder Spans Carrying Uniformly Distributed Live and Dead Loads.

If a girder without panels carries an electric railway, the end shear on the web and the reaction on the support can be figured in the same manner as was just explained for a railway girder, using the curves of Fig. 6*g*. The live-load shear at any section can be computed in the same manner as for a railway girder; but it will be found impossible to draw a curve similar to Fig. 21*b*, owing to the great irregularities in the wheel spacing. The moment can be figured as for a railway girder.

For girders without panels carrying road rollers or trucks, it will be necessary to figure the moments and shears directly.

When a girder with panels carries a load which is uniform per foot of span, but is concentrated on the girder at the various panel-points, the shears in the various panels are to be computed as for a truss span. It will be sufficiently accurate to use full panel loads. If the panel lengths are equal, Table 10*b* may be utilized to advantage. The maximum shear on the web is usually the shear in the end panel, although the load from the end floor-beam may sometimes have to be figured in. If the girder be riveted to other steelwork—as to a column—the maximum load on the end connection of the girder in most cases will be the same as the end shear on the web, although the load from the end floor-beam is also to be considered in computing the load on the column. If the girder rests on a shoe, the end stiffeners will generally have to carry the load from the end floor-beam as well as the end shear on the web. The manner in which this floor-beam load is transferred to the stiffeners must be duly considered, however. The reaction on the shoe will practically always include the load from the end floor-beam. The moment diagram is no longer a parabola, but is a polygon, the apices of which are at the panel-points. These apices lie on the parabola which would be the moment diagram of the girder if the load were uniformly distributed instead of concentrated. In order to draw the moment diagram, therefore, we proceed as follows. We first figure the centre moment M as though the load were uniformly distributed, employing the equation,

$$M = \tfrac{1}{8}wl^2, \qquad \text{[Eq. 1]}$$

where l is the span length and w the load per lineal unit. The span length is then laid out to any convenient scale, and a parabola is plotted with the above value of M as the central ordinate and the ordinates at the ends of the span equal to zero. Then the positions of the various panel-points are laid out and a vertical is erected at each point, cutting the moment parabola. The points where these verticals intersect the curve are then joined by straight lines, which form the moment diagram.

A girder with all of the load concentrated at the panel-points does not occur in actual practice, since the weight of the girder itself is always uniformly distributed. The effect of this is usually small, and should be neglected. When, however, a strip of flooring rests directly on the girder,

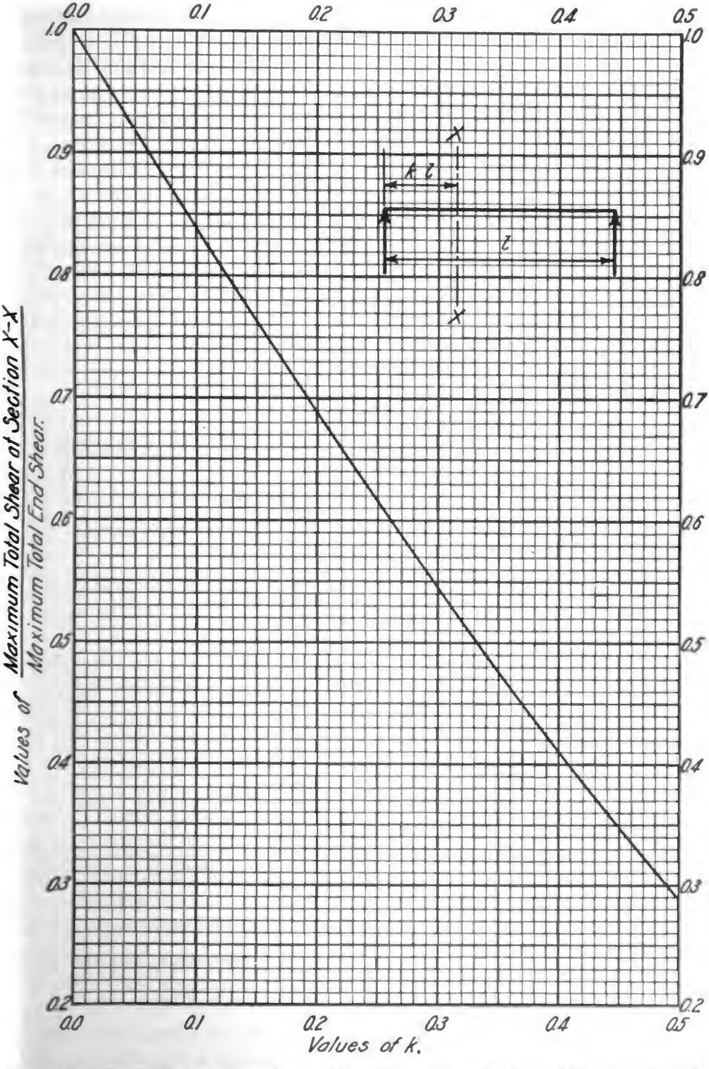


Fig. 21b. Total Shears throughout Plate-girder Spans without Floor-beams and Stringers Carrying Railway Loading.

it may be necessary to take into account the effect of this uniform load. In this case, the end shears due to uniform dead load, concentrated dead load, uniform live load, and concentrated live load should be figured separately. The shear at any section from each of the four loads can then be easily calculated. The moment diagram is drawn by first plotting the moment parabola, then constructing the moment polygon as though all of the load were concentrated at the panels, and lastly interpolating to get the correct moment diagram. For instance, if the uniform load were one-quarter of the total load, there should be located at the centre of each panel a point one-quarter of the way between the straight line and the parabola. The distance is to be measured vertically, not at right angles to the straight line. Then curves are drawn through these points and the adjacent panel-points on either side. If the panels are very long or the vertical distances quite large, it may be best to locate other points one-quarter of the way between the straight line and the parabola (measuring vertically); but in nearly every case the single point at the centre of each panel will suffice.

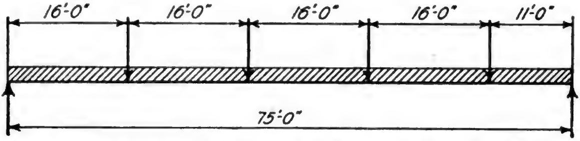


FIG. 21*c*. Layout of a 75-foot Girder with Varying Panel Lengths.

The above method of constructing the moment polygon is applicable whether the panel lengths are uniform or not. To illustrate clearly its application, let it be required to construct the moment diagram for a girder 75′ long from centre to centre of bearings, shown in Fig. 21*c*, having four panels each 16′ long and one panel 11′ in length, and carrying a total load of 6730 pounds per lineal foot, of which 2500 pounds is uniformly distributed along the girder, the remaining portion being carried by cross-girders at the various panel points. The moment at the centre is first figured on the assumption that all of the load is uniformly distributed, giving the result,

$$M = \tfrac{1}{8} \times 6730 \times 75^2 = 4{,}732{,}000 \text{ foot-pounds.}$$

The span length is then laid off to any convenient scale, and the above moment M plotted at the centre to any other convenient scale. See Fig. 21*d*. The moment parabola is then plotted. To do this, a horizontal line is drawn through the point already located at the centre, and ordinates are plotted downward therefrom at as many points as may be necessary. It will usually be best to divide each half span into four equal parts, in which case the ordinates at the various division points will be $1/16M$,

$\frac{1}{4}M$, and $9/16M$; or into five equal parts, in which case the said ordinates will be $1/25M$ ($= 0.04M$), $4/25M$ ($= 0.16M$), $9/25M$ ($= 0.36M$), and $16/25M$ ($= 0.64)M$; or into ten equal parts, in which case they will be $0.01M$, $0.04M$, $0.09M$, $0.16M$, $0.25M$, $0.36M$, $0.49M$, $0.64M$, and $0.81M$. (The parabola can also be drawn by any one of the several graphical methods.) In the present case, the half span was divided into five equal parts. For girders having panels of equal length, it might be better to plot simply the ordinates at each panel-point. For instance, in a seven-panel girder the panel-points are distant $1/7 \times l/2$, $3/7 \times l/2$, and $5/7 \times l/2$ from the mid-span point, and the corresponding ordinates are $1/49M$, $9/49M$, and $25/49M$.

The panel-points are then laid out, vertical lines are drawn at each point cutting the moment parabola, and the points of intersection are joined by straight lines. The resulting polygon would be the moment diagram if all of the load were concentrated at the panel-points. Since 2500 pounds of the total 6730 pounds are uniformly distributed, the vertical distance between the curve and the straight line is measured at the centre of each panel (this is the same for all of the 16′ panels) and $\frac{2500}{6730}$ of this distance laid off upward from the straight line. A smooth curve is then drawn in each panel through the point thus located and the two adjacent panel-points. The resulting figure is the moment diagram.

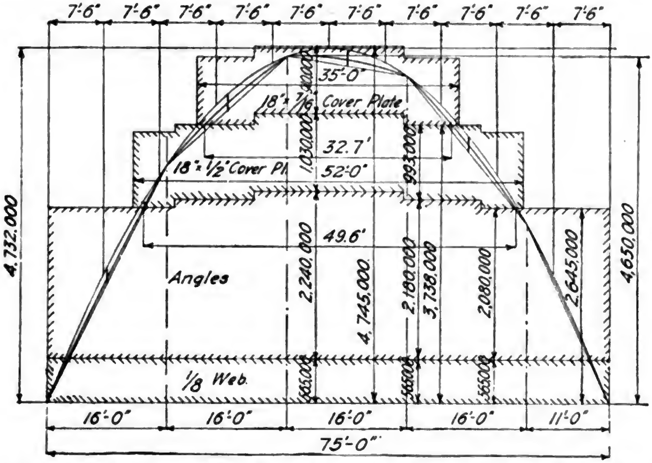


FIG. 21*d*. Moment Diagram for a 75-foot Girder with Varying Panel Lengths.

It will be found advisable to put the said diagram directly on the calculation blanks along with the other computations for the girder, rather than on a separate sheet of drawing paper. A scale of ten feet to the inch will generally be most suitable for laying out the dimensions of the girder. For plotting the moments, a scale of 1,000,000 foot-pounds to the inch should be used ordinarily, and one of 2,000,000 foot-pounds to the inch for very heavy girders.

For railway plate-girders with panels, such as the ordinary half-through girder, the maximum live-load end-shear on the web can be calculated by means of the engine diagram (such as given in Table 10*a*), if desired; but it will be sufficiently accurate to take from the curves of Fig. 6*c* the end shear for a span of length equal to the actual span minus eight-tenths (0.8) of the end panel, and then multiply this reading by the ratio of the reduced length to the actual length. For a girder of length l with short panels (say 12′ or less in length), the live-load shear at any panel-point distant kl from one end of the girder, due to load on the portion kl, can be taken as k times the end shear for a span of length kl. For panels over 12′ long, the length kl should be taken about one-tenth of a panel longer than the distance from the support to the panel-point in question. The maximum reaction on the support will usually be the same as for a girder without panels; but it may sometimes be equal to the maximum end-shear on the web.

The preceding discussion on finding moments and shears throughout girders has applied only to girders simply supported at both ends. Other types will now be discussed briefly.

Cantilever plate-girders are frequently used in steel construction. In a cantilever the shear at any point is equal to the sum of the loads outside of the section, and the moment is simply the moment of these loads about the point in question. If the girder carries uniform loading, the shear varies uniformly from zero at the end to a maximum at the support, while the moment varies as the ordinates of a parabola. If it carries concentrated loads, the shear is constant from load point to load point, and the moment curve between load points becomes a straight line. It will usually be necessary to figure the shear and the moment at each load point directly, as the loads and spacing in most cases will be irregular.

A cantilever is nearly always continuous with another girder. In designing the other girder, the effects on the moments and shears produced by loads on the cantilever must be duly considered. These loads increase the shear in the girder at the end next to the cantilever, and tend to cause an uplift at the other end. They usually tend to reverse the flange stress in the girder, so that it is necessary to figure the moment therein for the two following conditions of loading:

1. Live load and dead load on girder, and dead load only on cantilever.
2. Dead load only on girder, and live load and dead load on cantilever.

Plate-girders are frequently more or less continuous over several supports, but they are nearly always figured as simple beams. This assumption makes the work of design much easier, and is generally on the safe side. Of course, the formulæ for continuous beams can be applied to such girders, if it be thought advisable; but this should hardly ever be necessary.

The computation of the shears and moments at various sections of a girder is comparatively simple. The design of the various portions of the girder to care for these stresses in an effective and economical manner is much more difficult; and facility in such work can be attained only by experience. The remainder of this chapter will be devoted almost entirely to this subject.

Usually the first step in the design of a girder is the selection of the web plate. In order to do this, the maximum shear on the web must first be figured. It is also necessary to know the economic depth for the girder; and, furthermore, there are various practical points to be considered.

It was previously mentioned that the web is generally assumed to resist the entire shear, and in most cases this is practically correct; but in shallow plate girders with deep flanges the latter certainly help materially to resist shear. The error involved by the assumption, however, is on the side of safety.

In determining the required area for shear it is not necessary to figure the net section through a vertical line of rivets; for the intensity allowed in the specifications of Chapter LXXVIII is so safe that the gross area of the web may be employed instead of the net area. This saves a little trouble for the designer by simplifying his calculations. It will nearly always be found that, if an economic depth of girder be used, the minimum thickness of web permitted by the specifications or that is in conformity with good judgment or with established custom will provide a sectional area in excess of that required for maximum shear.

The question of economic depths for plate-girders is treated at length in Chapter LIII. Fig. 21*e* gives values for various span lengths and various total loads per lineal foot of span. The depths there indicated are really the effective depths rather than the depths of the web plates; but ordinarily the two figures are practically the same. Where no other factors control, the economic depth should be used. The web should, preferably, be so deep that the minimum permissible thickness specified in Chapter LXXVIII will furnish sufficient area for the shear, and will not require the flange rivets to be too closely spaced. The economic depths given in Fig. 21*e* will practically always secure these results; but where the clearance beneath the structure is limited, it is frequently necessary to adopt shallower girders. It will be found that a considerably smaller depth than the strictly economic one can be used with very little sacrifice of economy, unless the shear require an increase in the web thickness because of the smaller depth.

The depths of webs should be in full inches, if practicable—the adherence to even feet or half feet being preferable. The distance from out to out of flange angles should be made at least one-quarter (¼) of an inch greater than the depth of web, and for wide webs three-eighths (⅜) or even one-half (½) of an inch would be better. The reason for this is that the edges of webs are not always rolled or cut perfectly true; hence, if no variation were allowed for, they would project in places beyond the flange angles, thus either necessitating chipping or else giving an unworkmanlike finish.

Eleven feet is about as deep a girder as can be shipped over most railroads, and no depth exceeding that amount should be used without investigating the clearances of the roads over which the girder must pass. Furthermore, the fabrication of such deep girders is difficult and expensive.

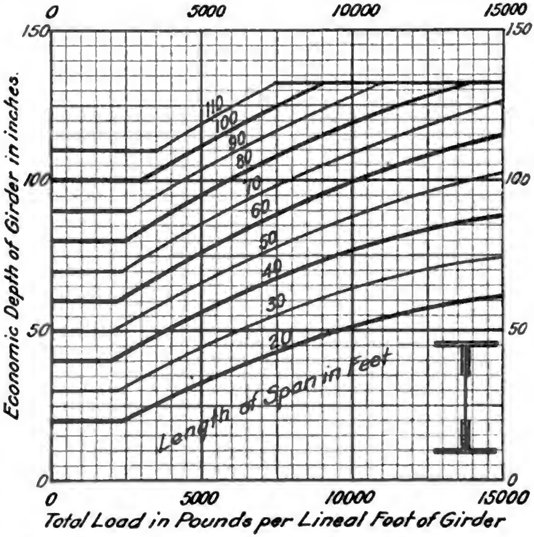


FIG. 21e. Economic Depths of Plate-girders with Riveted End-connections.

For girders over forty or forty-five feet long, it will be necessary to use one or more splices in the web. The designing of these splices is discussed later in this chapter. The limiting dimensions of plates are given in the hand-books of the various steel manufacturers—for instance, in Carnegie's Pocket Companion. The number of splices should be as small as possible, notwithstanding the fact that extra large plates command a higher pound price than ordinary bridge metal. There are two sound objections to using short and light component pieces in the manufacture of plate-girder spans: first, the additional metal required for the splicing; and, second, the loss of strength in the splices, notwithstanding every effort that can be made by designing scientifically to reduce that loss to a minimum.

Much has been written concerning the theory of stress distribution in the webs of plate-girders and on the proper location of the intermediate stiffeners; but all such theories are upset by the fact that many plate-girders having stiffeners only over the supports have carried heavy loads for years without showing any sign of buckling or overstress. This is no good reason, though, for the omission of these details in plate-girder designing; for, in addition to their theoretical functions, they are useful in supporting the top flanges in deck spans, and they keep the web from injury by bending during manufacture, transportation, and erection. The economics of crimping stiffeners is treated fully in Chapter LIII. The importance of having stiffeners fit tightly against the flange angles of deck girders at both top and bottom cannot be too strongly emphasized. It really would not be important to have a tight fit at bottom, were it not for the fact that tightness at top is dependent thereon; therefore the stiffeners should go into place with a driving fit. The specifications for the spacing and sizes of intermediate stiffeners are given in Chapter LXXVIII.

There are two methods of proportioning a girder to resist a given bending moment. One is by the use of the moment of inertia or section modulus of the entire section. If any rivet holes exist in the portion of the section which is under tension, the net moment of inertia is to be employed instead of the gross, it being customary to consider the moment of inertia of the compression side equal to that of the tension side. In the other method, all of each flange area and one-eighth of the web area are assumed to be concentrated at the center of gravity of each flange, and the bending moment is considered to be resisted wholly by these two areas. The first of these methods is used for the design of I-beams and channels, or other structural shapes; while the second method is almost always employed for plate-girders. There is a tendency for the extreme fibres of girders designed on this latter basis to have somewhat higher stresses than this average stress in the flange. In deep girders this excess is not appreciable; but in very shallow built-up girders it must be taken into consideration, and such girders should be designed by means of the moment of inertia.

The method of assuming that one-eighth ($\frac{1}{8}$) of the area of the web is concentrated in each flange to resist bending is a very close approximation to correctness. If there were no rivet holes at all in the web (which is, of course, impossible), the allowance would be one-sixth ($\frac{1}{6}$), and in cases where the webs are very badly chopped up by the rivet holes for attaching the intermediate stiffeners or the splice plates, it might go as low as one-ninth ($\frac{1}{9}$).

Splices in flanges should be dispensed with, if possible; but if they be unavoidable, such splices should be staggered both with each other and with the web splices in order that the joints in all the various component parts of the girder may be separated so far that the zones of compensation for the weakening effect of the splicing will never overlap. A good location for a joint in a flange component is just beyond the point where a cover-plate would otherwise end; because by continuing the said cover-plate beyond that point it will suffice for a reinforcing plate, provided that its sectional area be great enough. If not, an additional short cover-plate or splice-angle will have to be employed. It is rarely necessary to splice flange angles, 8″ × 8″ angles 130 feet long having been secured for some recent girders.

The simplest form of flange consists of two angles, as shown in Fig. 21*f*. This type should generally be adopted wherever sufficient sectional area can be obtained without the use of unduly heavy angles—say 6″ × 6″ × ¾″ as a maximum. It is true that the employment of cover-plates and angles for smaller sections will save a small amount of metal, but considerable additional riveting will be required, which will offset a portion of the saving; and, in the case of girders carrying ties on the top flanges, the rivet heads in the horizontal legs are undesirable. For light sections, in which angles smaller than 6″ × 6″ are employed, it will be necessary to test the rivet pitch required at the ends of the girder, as frequently a pitch considerably less than 3″ will be needed, and then the adoption of a double gauge-line is obligatory. For such sections two 5″ × 3½″, 6″ × 3½″, or 6″ × 4″ angles with the shorter legs outstanding should be used. Where the single gauge-line is sufficient, it will be well to employ unequal-legged angles with the longer legs out, as thereby the bracing points can be placed farther apart, or, in case that the positions of bracing points are fixed, the allowable stress in the compression flange can be increased.

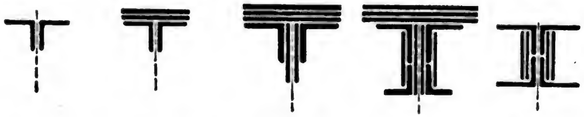


Fig. 21*f*. Fig. 21*g*. Fig. 21*h*. Fig. 21*i*. Fig. 21*j*.

Typical Flange Sections.

For flanges in which two 6″ × 6″ × ¾″ angles fail to give sufficient area, the section most generally used consists of two angles with cover-plates, as shown in Fig. 21*g*. Two 6″ × 6″ angles with cover-plates 14″ wide should be employed for lighter sections, and two 8″ × 8″ angles with cover-plates 18″ or 20″ wide for heavier sections. When two 8″ × 8″ × 1⅛″ angles fail to give a sufficient proportion of the area in the angles, two vertical side-plates under the angles should be adopted, thus producing the form of flange shown in Fig. 21*h*. Ordinarily the width of these side-plates should be 12″, but for very heavy flanges they may have to be wider. The cover-plates may be made as wide as 24″, if necessary. If still more section should be required (which would very rarely happen), the type of flange shown in Fig. 21*i* could be used. It would be best to make the angles which rivet to the cover plates 8″ × 8″ × 1″, and the others 6″ × 6″ × 1″; however, four 8″ × 8″ angles could be employed, if necessary.

Table 21*a* gives good working limits for the various types of flanges.

TABLE 21*a*

LIMITING SECTIONS FOR VARIOUS TYPES OF GIRDER FLANGES

Type of Section	Maximum or Minimum	Dimensions of Angles and Plates	Gross Section in Sq. In.	Net Section in Sq. In., 2-1″ Holes Out of Each Angle and Plate
2 angles........	Max.	2 ∟s 6″×6″×¾″	16.88	13.88
2–6″×6″ ∟s .. 14″ cover-plates.	Min.	2 ∟s 6″×6″×½″ 1 cover-plate 14″×⅜″	11.50 5.25	9.50 4.50
			16.75	14.00
	Max.	2 ∟s 6″×6″×13/16″ 2 cover-plates 14″×⅝″	18.18 17.50	14.93 15.00
			35.68	29.93
2–8″×8″ ∟s .. 18″ cover-plates.	Min.	2 ∟s 8″×8″×⅝″ 2 cover-plates 18″×7/16″	19.22 15.75	16.72 14.00
			34.97	30.72
	Max.	2 ∟s 8″×8″×1⅛″ 3 cover-plates 18″×¾″	33.46 40.50	28.94 36.00
			73.96	64.94
2–8″×8″ ∟s .. 20″ cover-plates.	Min.	2 ∟s 8″×8″×1⅛″ 3 cover-plates 20″×11/16″	33.46 41.25	28.94 37.12
			74.71	66.06
	Max.	2 ∟s 8″×8″×1⅛″ 3 cover-plates 20″×13/16″	33.46 48.75	28.94 43.88
			82.21	72.82

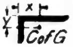
A thickness of 13/16″ is selected as the maximum for the 6″ × 6″ angles with cover-plates, as thicker metal will have to be drilled; and in many cases it would be best to make the 6″ × 6″ × ¾″ angle the limit, as 13/16″ metal is a trifle heavy for punching. The area of cover-plates is taken the same as that of the angles, for this proportion should, preferably, not be exceeded. The 8″ × 8″ × ⅝″ angle is the thinnest 8″ × 8″ angle that should ever be used in a compression flange, unless there be a full-length cover-plate; and generally 11/16″ should be the minimum, as the ⅝″ thickness is a trifle under the specification requirement. The

TABLE

CENTRES OF GRAVITY AND GROSS AND NET

DISTANCE OF CENTRE OF GRAVITY FROM BACK OF ANGLE IN INCHES

SIZE	3/8		1/2		5/8		3/4		7/8		1		1 1/8	
	x	*y*	*x*	*y*	*x*	*y*	*x*	*y*	*x*	*y*	*x*	*y*	*x*	*y*
8 × 8	.....	.....	2.19	.....	2.23	.....	2.28	.....	2.32	.....	2.37	.....	2.41	.....
6 × 6	1.64	.....	1.68	.....	1.73	.....	1.78	.....	1.82	.....	1.86	.....	.....	.....
5 × 5†	1.39	.....	1.43	.....	1.48	.....	1.52	.....	1.57	.....	1.61	.....	.....	.....
4 × 4	1.14	.....	1.18	.....	1.23	.....	1.27	.....	.....	.....	.....	.....	.....	.....
3½ × 3½	1.01	.....	1.06	.....	1.10	.....	1.15	.....	.....	.....	.....	.....	.....	.....
3 × 3	0.89	.....	0.93	.....	0.98	.....	.....	.....	.....	.....	.....	.....	.....	.....
8 × 6	.....	.....	2.47	1.47	2.52	1.52	2.56	1.56	2.61	1.61	2.65	1.65	.....	.....
7 × 3½	.....	.....	2.53	0.78	2.57	0.82	2.62	0.87	2.66	0.91	2.70	0.96	.....	.....
6 × 4	1.94	0.94	1.99	0.99	2.03	1.03	2.08	1.08	2.12	1.12	2.17	1.17	.....	.....
6 × 3½	2.04	0.78	2.08	0.83	2.13	0.88	2.18	0.93	2.22	0.97	2.26	1.01	.....	.....
5 × 4†	1.53	1.03	1.57	1.07	1.62	1.12	1.66	1.16	1.71	1.21	.....	.....	.....	.....
5 × 3½	1.61	0.86	1.66	0.91	1.70	0.95	1.75	1.00	1.79	1.04	.....	.....	.....	.....
5 × 3	1.70	0.70	1.75	0.75	1.80	0.80	1.84	0.84	.....	.....	.....	.....	.....	.....
4 × 3½	1.21	0.96	1.25	1.00	1.29	1.04	1.34	1.09	.....	.....	.....	.....	.....	.....
4 × 3	1.28	0.78	1.33	0.83	1.37	0.87	1.42	0.92	.....	.....	.....	.....	.....	.....
3½ × 3	1.08	0.83	1.13	0.88	1.17	0.92	1.21	0.96	.....	.....	.....	.....	.....	.....



GROSS AREAS OF 2 ANGLES IN SQUARE INCHES

SIZE	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8
8 × 8	.....	.....	.....	15.50	17.36	19.22	21.06	22.88	24.68	26.46	28.24	30.00	31.74	33.46
6 × 6	.....	8.72	10.12	11.50	12.86	14.22	15.56	16.88	18.18	19.48	20.74	22.00	.....	.....
5 × 5†	.....	7.22	8.36	9.50	10.62	11.72	12.80	13.88	14.94	15.96	17.00	18.00	.....	.....
4 × 4	4.80	5.72	6.62	7.50	8.36	9.22	10.06	10.88	11.68	.....	.....	.....	.....	.....
3½ × 3½	4.18	4.96	5.74	6.50	7.24	7.96	8.68	9.38	10.06	.....	.....	.....	.....	.....
3 × 3	3.56	4.22	4.86	5.50	6.12	6.72	.....	.....	.....	.....	.....	.....	.....	.....
8 × 6	.....	.....	.....	13.50	15.12	16.72	18.30	19.88	21.44	22.96	24.50	26.00	.....	.....
7 × 3½	.....	.....	8.80	10.00	11.18	12.34	13.50	14.62	15.74	16.84	17.94	19.00	.....	.....
6 × 4	.....	7.22	8.36	9.50	10.62	11.72	12.80	13.88	14.94	15.96	17.00	18.00	.....	.....
6 × 3½	.....	6.82	7.94	9.00	10.06	11.10	12.12	13.12	14.12	15.10	16.06	17.00	.....	.....
5 × 4†	.....	6.46	7.50	8.50	9.50	10.46	11.44	12.38	13.30	14.22	.....	.....	.....	.....
5 × 3½	5.12	6.10	7.06	8.00	8.94	9.84	10.74	11.62	12.50	13.34	.....	.....	.....	.....
5 × 3	4.80	5.72	6.62	7.50	8.36	9.22	10.06	10.88	11.68	.....	.....	.....	.....	.....
4 × 3½	4.50	5.34	6.18	7.00	7.80	8.60	9.36	10.12	10.86	.....	.....	.....	.....	.....
4 × 3	4.18	4.96	5.74	6.50	7.24	7.96	8.68	9.38	10.06	.....	.....	.....	.....	.....
3½ × 3	3.86	4.60	5.30	6.00	6.68	7.34	8.00	8.62	9.24	.....	.....	.....	.....	.....

† Difficult to obtain.

21*b*

AREAS OF ANGLES FOR GIRDER FLANGES

NET AREAS OF 2 ANGLES IN SQUARE INCHES—2 1-INCH HOLES DEDUCTED

SIZE	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8
8 × 8	.....	.....	.....	14.50	16.23	17.97	19.68	21.38	23.05	24.71	26.36	28.00	29.62	31.21
6 × 6	.....	7.97	9.24	10.50	11.73	12.97	14.18	15.38	16.55	17.73	18.86	20.00	.....	.....
5 × 5†	.....	6.47	7.48	8.50	9.50	10.47	11.43	12.38	13.31	14.21	15.12	16.00	.....	.....
4 × 4	4.17	4.97	5.74	6.50	7.23	7.97	8.68	9.38	10.05	.....	.....	.....	.....	.....
3½ × 3½	3.55	4.21	4.86	5.50	6.11	6.71	7.30	7.88	8.43	.....	.....	.....	.....	.....
3 × 3	2.93	3.47	3.98	4.50	5.00	5.47	.....	.....	.....	.....	.....	.....	.....	.....
8 × 6	.....	.....	.....	12.50	14.00	15.47	16.92	18.38	19.82	21.21	22.62	24.00	.....	.....
7 × 3½	.....	.....	7.92	9.00	10.05	11.09	12.12	13.12	14.11	15.09	16.06	17.00	.....	.....
6 × 4	.....	6.47	7.48	8.50	9.50	10.47	11.42	12.38	13.31	14.21	15.12	16.00	.....	.....
6 × 3½	.....	6.07	7.06	8.00	8.93	9.85	10.74	11.62	12.49	13.35	14.18	15.00	.....	.....
5 × 4†	.....	5.71	6.62	7.50	8.37	9.21	10.06	10.88	11.67	12.47	.....	.....	.....	.....
5 × 3½	4.49	5.35	6.18	7.00	7.81	8.59	9.36	10.12	10.87	11.59	.....	.....	.....	.....
5 × 3	4.17	4.97	5.74	6.50	7.23	7.97	8.68	9.38	10.05	.....	.....	.....	.....	.....
4 × 3½	3.87	4.59	5.30	6.00	6.67	7.35	7.98	8.62	9.23	.....	.....	.....	.....	.....
4 × 3	3.55	4.21	4.86	5.50	6.11	6.71	7.30	7.88	8.43	.....	.....	.....	.....	.....
3½ × 3	3.23	3.85	4.42	5.00	5.55	6.09	6.62	7.12	7.61	.....	.....	.....	.....	.....

NET AREAS OF 2 ANGLES IN SQUARE INCHES—4 1-INCH HOLES DEDUCTED

SIZE	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8
8 × 8	.....	.....	.....	13.50	15.11	16.72	18.31	19.88	21.43	22.96	24.49	26.00	27.50	28.94
6 × 6	.....	7.22	8.37	9.50	10.61	11.72	12.81	13.88	14.93	15.98	17.00	18.00	.....	.....
5 × 5†	.....	5.72	6.61	7.50	8.37	9.22	10.05	10.88	11.69	12.46	13.25	14.00	.....	.....
4 × 4	3.55	4.22	4.87	5.50	6.11	6.72	7.31	7.88	8.43	.....	.....	.....	.....	.....
3½ × 3½	2.93	3.46	3.99	4.50	4.99	5.46	5.93	6.38	6.81	.....	.....	.....	.....	.....
3 × 3	2.31	2.72	3.11	3.50	3.87	4.22	.....	.....	.....	.....	.....	.....	.....	.....
8 × 6	.....	.....	.....	11.50	12.88	14.20	15.55	16.88	18.20	19.46	20.75	22.00	.....	.....
7 × 3½	.....	.....	7.05	8.00	8.93	9.84	10.75	11.62	12.49	13.34	14.19	15.00	.....	.....
6 × 4	.....	5.72	6.61	7.50	8.37	9.22	10.05	10.88	11.69	12.46	13.25	14.00	.....	.....
6 × 3½	.....	5.32	6.19	7.00	7.81	8.60	9.37	10.12	10.87	11.60	12.31	13.00	.....	.....
5 × 4†	.....	4.96	5.75	6.50	7.25	7.96	8.69	9.38	10.05	10.72	.....	.....	.....	.....
5 × 3½	3.87	4.60	5.31	6.00	6.69	7.34	7.99	8.62	9.25	9.84	.....	.....	.....	.....
5 × 3	3.55	4.22	4.87	5.50	6.11	6.72	7.31	7.88	8.43	.....	.....	.....	.....	.....
4 × 3½	3.25	3.84	4.43	5.00	5.55	6.10	6.61	7.12	7.61	.....	.....	.....	.....	.....
4 × 3	2.93	3.46	3.99	4.50	4.99	5.46	5.93	6.38	6.81	.....	.....	.....	.....	.....
3½ × 3	2.61	3.10	3.55	4.00	4.43	4.84	5.25	5.62	5.99	.....	.....	.....	.....	.....

† Difficult to obtain.

TABLE 21c

COVER-PLATES FOR GIRDERS
AREAS AND WEIGHTS PER LINEAL FOOT

Thickness, Inches	14-INCH			16-INCH			18-INCH			20-INCH		
	Wt. per Foot Lbs.	AREA SQ. INS.		Wt. per Foot Lbs.	AREA SQ. INS.		Wt. per Foot Lbs.	AREA SQ. INS.		Wt. per Foot Lbs.	AREA SQ. INS.	
		Gross	Net, 2 1″-Holes Out		Gross	Net, 2 1″-Holes Out		Gross	Net, 2 1″-Holes Out		Gross	Net, 2 1″-Holes Out
1/16	2.97	0.875	0.75	3.40	1.000	0.875	3.83	1.125	1.00	4.25	1.25	1.125
1/8	5.95	1.750	1.50	6.80	2.000	1.750	7.65	2.250	2.00	8.50	2.50	2.250
3/16	8.92	2.625	2.25	10.20	3.000	2.625	11.48	3.375	3.00	12.75	3.75	3.375
1/4	11.90	3.500	3.00	13.60	4.000	3.500	15.30	4.500	4.00	17.00	5.00	4.500
5/16	14.88	4.375	3.75	17.00	5.000	4.375	19.12	5.625	5.00	21.25	6.25	5.625
3/8	17.85	5.250	4.50	20.40	6.000	5.250	22.95	6.750	6.00	25.50	7.50	6.750
7/16	20.82	6.125	5.25	23.80	7.000	6.125	26.78	7.875	7.00	29.75	8.75	7.875
1/2	23.80	7.000	6.00	27.20	8.000	7.000	30.60	9.000	8.00	34.00	10.00	9.000
9/16	26.78	7.875	6.75	30.60	9.000	7.875	34.43	10.125	9.00	38.25	11.25	10.125
5/8	29.75	8.750	7.50	34.00	10.000	8.750	38.25	11.250	10.00	42.50	12.50	11.250
11/16	32.72	9.625	8.25	37.40	11.000	9.625	42.08	12.375	11.00	46.75	13.75	12.375
3/4	35.70	10.500	9.00	40.80	12.000	10.500	45.90	13.500	12.00	51.00	15.00	13.500
13/16	38.67	11.375	9.75	44.20	13.000	11.375	49.72	14.625	13.00	55.25	16.25	14.625
7/8	41.65	12.250	10.50	47.60	14.000	12.250	53.55	15.750	14.00	59.50	17.50	15.750
15/16	44.63	13.125	11.25	51.00	15.000	13.125	57.38	16.875	15.00	63.75	18.75	16.875
1	47.60	14.000	12.00	54.40	16.000	14.000	61.20	18.000	16.00	68.00	20.00	18.000
1 1/16	50.57	14.875	12.75	57.80	17.000	14.875	65.02	19.125	17.00	72.25	21.25	19.125
1 1/8	53.55	15.750	13.50	61.20	18.000	15.750	68.85	20.250	18.00	76.50	22.50	20.250
1 3/16	56.52	16.625	14.25	64.60	19.000	16.625	72.68	21.375	19.00	80.75	23.75	21.375
1 1/4	59.50	17.500	15.00	68.00	20.000	17.500	76.50	22.500	20.00	85.00	25.00	22.500
1 5/16	62.47	18.375	15.75	71.40	21.000	18.375	80.33	23.625	21.00	89.25	26.25	23.625
1 3/8	65.45	19.250	16.50	74.80	22.000	19.250	84.15	24.750	22.00	93.50	27.50	24.750
1 7/16	68.42	20.125	17.25	78.20	23.000	20.125	87.98	25.875	23.00	97.75	28.75	25.875
1 1/2	71.40	21.000	18.00	81.60	24.000	21.000	91.80	27.000	24.00	102.00	30.00	27.000
1 9/16	74.37	21.875	18.75	85.00	25.000	21.875	95.63	28.125	25.00	106.25	31.25	28.125
1 5/8	77.35	22.750	19.50	88.40	26.000	22.750	99.45	29.250	26.00	110.50	32.50	29.250
1 11/16	80.32	23.625	20.25	91.80	27.000	23.625	103.28	30.375	27.00	114.75	33.75	30.375
1 3/4	83.30	24.500	21.00	95.20	28.000	24.500	107.10	31.500	28.00	119.00	35.00	31.500
1 13/16	86.27	25.375	21.75	98.60	29.000	25.375	110.93	32.625	29.00	123.00	36.25	32.625
1 7/8	89.25	26.250	22.50	102.00	30.000	26.250	114.75	33.750	30.00	127.50	37.50	33.750
1 15/16	92.22	27.125	23.25	105.40	31.000	27.125	118.58	34.875	31.00	131.75	38.75	34.875
2	95.20	28.000	24.00	108.80	32.000	28.000	122.40	36.000	32.00	136.00	40.00	36.000
2 1/16	98.17	28.875	24.75	112.20	33.000	28.875	126.23	37.125	33.00	140.25	41.25	37.125
2 1/8	101.15	29.750	25.50	115.60	34.000	29.750	130.05	38.250	34.00	144.50	42.50	38.250
2 3/16	104.12	30.625	26.25	119.00	35.000	30.625	133.88	39.375	35.00	148.75	43.75	39.375
2 1/4	107.10	31.500	27.00	122.40	36.000	31.500	137.70	40.500	36.00	153.00	45.00	40.500
2 5/16	......	......	......	......	......	......	141.53	41.625	37.00	157.25	46.25	41.625
2 3/8	......	......	......	......	......	......	145.35	42.750	38.00	161.50	47.50	42.750
2 7/16	......	......	......	......	......	......	149.18	43.875	39.00	165.75	48.75	43.875
2 1/2	......	......	......	......	......	......	153.00	45.000	40.00	170.00	50.00	45.000
2 9/16	......	......	......	......	......	......	156.83	46.125	41.00	174.25	51.25	46.125
2 5/8	......	......	......	......	......	......	160.65	47.250	42.00	178.50	52.50	47.250
2 11/16	......	......	......	......	......	......	164.48	48.375	43.00	182.75	53.75	48.375
2 3/4	......	......	......	......	......	......	168.30	49.500	44.00	187.00	55.00	49.500

TABLE 21*d*

CENTRES OF GRAVITY OF GIRDER FLANGES COMPOSED OF TWO ANGLES WITH COVER-PLATES

DISTANCES OF CENTRES OF GRAVITY FROM BACKS OF ANGLES IN INCHES

Total Thickness of Plates, Inches	2 L° 6″×6″×½″		2 L° 6″×6″×⅝″		2 L° 6″×6″×¾″		2 L° 6″×6″×⅞″		2 L° 6″×6″×1″		2 L° 8″×8″×⅝″		2 L° 8″×8″×¾″		2 L° 8″×8″×⅞″		2 L° 8″×8″×1″		2 L° 8″×8″×1⅛″		Total Thickness of Plates, Inches
	Cover-Plate 14″	Cover-Plate 16″	Cover-Plate 14″	Cover-Plate 16″	Cover-Plate 14″	Cover-Plate 16″	Cover-Plate 14″	Cover-Plate 16″	Cover-Plate 14″	Cover-Plate 16″	Cover-Plate 18″	Cover-Plate 20″	Cover-Plate 18″	Cover-Plate 20″	Cover-Plate 18″	Cover-Plate 20″	Cover-Plate 18″	Cover-Plate 20″	Cover-Plate 18″	Cover-Plate 20″	
0	1.68	1.68	1.73	1.73	1.78	1.78	1.82	1.82	1.86	1.86	2.23	2.23	2.28	2.28	2.32	2.32	2.37	2.37	2.41	2.41	0
⅜	1.09	1.04	1.21	1.16	1.31	1.26	1.39	1.35	1.47	1.43	1.60	1.54	1.72	1.66	1.81	1.76	1.90	1.85	1.97	1.94	⅜
½	0.95	0.89	1.08	1.02	1.18	1.13	1.27	1.22	1.36	1.30	1.44	1.38	1.57	1.51	1.67	1.62	1.77	1.72	1.85	1.80	½
⅝	0.82	0.75	0.95	0.89	1.07	1.00	1.16	1.10	1.25	1.18	1.29	1.23	1.43	1.37	1.53	1.49	1.64	1.59	1.73	1.68	⅝
¾	0.70	0.63	0.83	0.77	0.95	0.88	1.05	0.98	1.14	1.07	1.15	1.09	1.29	1.24	1.41	1.36	1.52	1.46	1.61	1.56	¾
⅞	0.59	0.52	0.73	0.65	0.85	0.78	0.95	0.87	1.04	0.97	1.02	0.96	1.17	1.12	1.29	1.23	1.40	1.34	1.50	1.44	⅞
1	0.48	0.41	0.63	0.55	0.75	0.67	0.85	0.78	0.95	0.87	0.91	0.84	1.06	1.00	1.18	1.11	1.29	1.22	1.40	1.33	1
1⅛	0.39	....	0.53	0.45	0.65	0.59	0.76	0.68	0.86	0.77	0.80	0.73	0.95	0.88	1.07	1.00	1.18	1.11	1.30	1.22	1⅛
1¼	0.29	....	0.43	0.35	0.55	0.47	0.66	0.58	0.77	0.68	0.69	0.62	0.84	0.77	0.93	0.90	1.08	1.01	1.20	1.12	1¼
1⅜	....	....	0.34	....	0.46	0.38	0.57	0.49	0.68	0.59	0.59	0.52	0.74	0.67	0.87	0.80	0.98	0.91	1.10	1.02	1⅜
1½	....	....	0.25	....	0.38	0.29	0.49	0.40	0.59	0.50	0.49	....	0.64	0.57	0.77	0.70	0.89	0.81	1.01	0.92	1½
1⅝	....	....	....	....	0.29	....	0.40	0.32	0.50	0.41	....	....	0.54	0.47	0.67	0.60	0.80	0.71	0.92	0.83	1⅝
1¾	....	....	....	....	0.20	....	0.32	0.22	0.42	0.33	....	....	0.45	....	0.57	0.50	0.71	0.62	0.83	0.74	1¾
1⅞	....	....	....	....	....	....	0.24	....	0.34	0.25	....	....	0.36	....	0.49	0.41	0.62	0.53	0.74	0.64	1⅞
2	....	....	....	....	....	....	0.16	....	0.26	0.17	....	....	....	....	0.40	....	0.53	0.44	0.65	0.55	2
2⅛	....	....	....	....	....	....	....	....	0.19	....	....	....	....	....	0.32	....	0.44	0.36	0.56	0.47	2⅛
2¼	....	....	....	....	....	....	....	....	0.11	....	....	....	....	....	....	....	0.36	0.28	0.48	0.38	2¼
2⅜	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	0.28	....	0.40	0.30	2⅜
2½	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	0.20	....	0.32	0.22	2½
2⅝	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	0.24	....	2⅝
2¾	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	0.16	....	2¾

The values above the dotted lines are for combinations in which the areas of the cover-plates are less than 50 per cent of the totals; and those between the dotted lines and the full lines are for combinations in which the areas of the cover-plates are from 50 to 60 per cent of the totals.

8″ × 8″ × 1⅛″ angle is the maximum that can be obtained, and the maximum cover-plate area is governed by the requirement that not more than sixty (60) per cent of the flange section is to be in the cover-plates. Where three 18″ × ¾″ cover-plates fail to give sufficient area, the employment of cover-plates 20″ wide is to be preferred, in order to keep down the number of these, and also the total thickness. The piling up of cover-plates is bad practice; first, because the rivets become so long that they do not fill the holes properly, and second, because it is very difficult to distribute the stress uniformly over a section composed of a mass of cover-plates.

In order to facilitate the designing of flanges, Tables 21*b*, 21*c*, and 21*d*, and Fig. 21*k* have been prepared. Table 21*b* gives the position of the centre of gravity of any angle; and also, for two angles, the gross areas, the net areas with one one-inch hole out of each angle, and the net areas with two one-inch holes out of each angle. Table 21*c* gives, for various cover-plates, the weights per lineal foot, the gross areas, and the net areas with two one-inch holes out. Table 21*d* shows the positions of the centres of gravity of flanges composed of two angles with cover-plates. Fig. 21*k* shows directly the various combinations of angles and cover-plates that will give any desired net sectional area. The combinations for which one-half of the flange area will be in the cover-plates are indicated, as are also those for which sixty per cent thereof is in the said cover-plates.

Some engineers require that the first cover-plate of a flange shall extend the full length of the girder, but the author sees no good reason for such a requirement. The idea in it apparently is to distribute the load from the ties over both angles of the top flange, and then to make, as usual, both flanges alike; but it would be far better to support the top flanges effectively by using stiffeners with long outstanding angle-legs. It has been claimed by some that the cover-plate prevents the brine drippings of refrigerator cars from working down between the flange angles and the web; but the author has never seen any trouble of that nature from this cause, although the tops of the flanges are frequently rusted thereby.

When cover-plates fourteen, eighteen, or twenty inches wide are used, there should be four lines of rivets fastening them to the flange angles, those in the two inner rows being staggered with those in the two outer rows. When twenty-two or twenty-four inch cover-plates are adopted, this same riveting should be used, and, in addition, there should be a line of rivets along each edge of the cover-plates outside of the angles.

The fact that the rivet heads in the cover-plates are undesirable when ties rest directly on the top flange has already been mentioned. For this reason a number of railways specify that for bridges of this type cover-plates shall not be used for the top flanges. When designing girders to meet this requirement, light flanges should be made up of two angles; and when the required areas are greater than can be provided for in this manner, the bottom flange should consist of two angles with cover-plates,

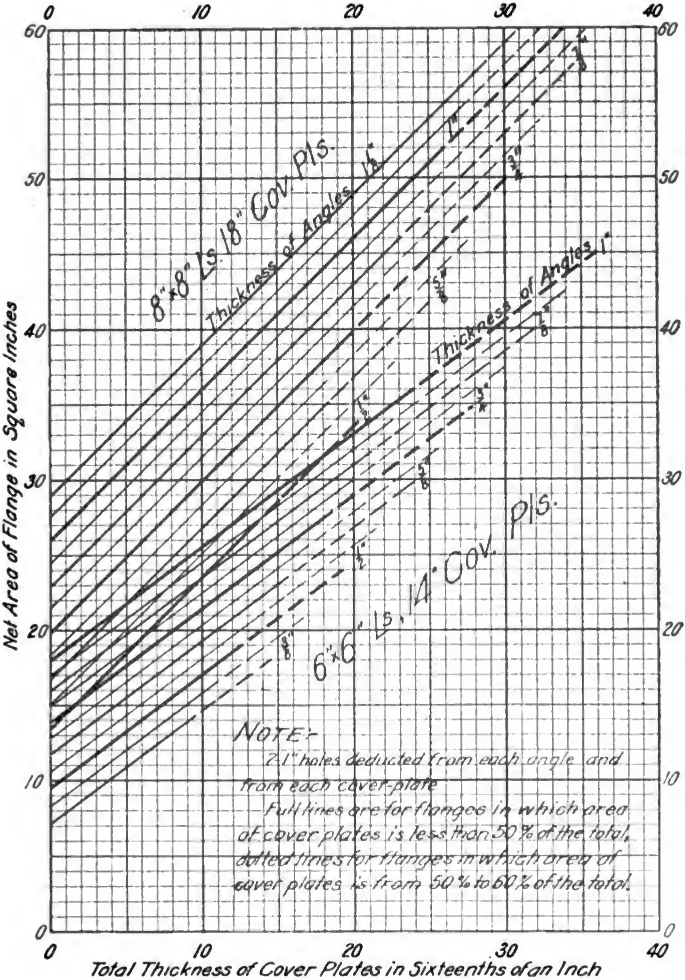


FIG. 21*k*. Net Areas of Plate-girder Flanges composed of Two Angles and Cover-plates.

and the top flange of four angles either with or without side-plates, arranged as shown in Fig. 21*j*. The distance from back to back of the two pairs of angles should be about one-half of an inch greater than the sum of the lengths of the two vertical legs, in order to provide for possible overruns of the angles. Table 21*e* shows the manner in which such flanges may be built up.

TABLE 21*e*

MAKE-UP OF FLANGES WITHOUT COVER-PLATES

Type of Section	Dimensions of Angles and Plates	Gross Section in Square Inches	Distance Back to Back of Angles in Inches
2 ∟s	2 ∟s 6″×6″×⅞″ maximum	19.48	
	2 ∟s 8″×6″×(¾″–⅞″)...	19.88–22.96	
	2 ∟s 8″×8″×(¾″–⅞″)...	22.88–26.46	
4 ∟s and side-plates	4 ∟s 6″×4″×(½″–¾″) 6½″ side-plates.........	19.00–27.76	8½″
	2 ∟s 6″×6″×(½″–¾″).. 2 ∟s 6″×4″×(½″–¾″) 8½″ side-plates.........	11.50–16.88 9.50–13.88	10½″
	4 ∟s 6″×6″×(11/16″–13/16″) 10½″ side-plates.........	31.12–36.36	12½″
	4 ∟s 8″×6″×(11/16″–1″) 10″ side-plates.........	36.60–52.00	12½″
	4 ∟s 8″×8″×(⅞″–1⅛″) 13½″ side-plates........	52.94–66.92	16½″

The use of very heavy angles for the two-angle flanges is justified in this case, as it allows both flanges to be alike, thus simplifying the shopwork, and also gives a greater effective depth for the same web than does the four-angle flange. The 8″ × 8″ angles should not be used for shallow girders, however. Whenever unequal-legged angles are adopted for either type of flange, the longer leg is to be outstanding. The 6″ × 4″ angles are preferable to the 6″ × 6″, as for the same web the effective depth is greater. However, it will sometimes be impossible to place enough rivets in the upper pair of a four-angle flange when the 6″ × 4″ angles are used, and in that case the 6″ × 6″ angles must be employed, keeping the lower angles 6″ × 4″ if possible. The 8″ × 6″ angles are preferable to the 8″ × 8″, as they make the effective depth greater.

With flanges of the type shown in Figs. 21*i* and 21*j* it is impossible to make the stiffening angles continuous for the full depth of the web, for they must be cut at the inner angles of each flange, as shown in Fig. 21*w*. The stiffeners must fit tightly between the angles of the flanges. It is the author's practice to insert short stiffeners in the top flange midway between consecutive main intermediate stiffeners, so as to support the ties effectively. Owing to the discontinuity of all of the stiffeners, there is a horizontal line of weakness in the web just inside of the inner pair of angles in each flange; but the detail of the sway frames shown in Fig. 21*w*, which the author always employs for girders with this type of flange, will overcome this weakness, if it really exists.

As the allowable stress in the compression flange is dependent on the ratio $\frac{l}{b}$, where l is the distance between the points where the flange is held in line transversely and b is the flange width, economy will evidently be secured by bracing it at rather short intervals. It is further required by the specifications of Chapter LXXVIII, however, that the section of the compression flange shall not be less than the gross section of the tension flange. Table 21*f* gives, for flanges of various types, the ratios $\frac{l}{b}$ and lengths l at which the required area of the compression flange will about equal the gross area of the tension flange. The values are approximate only, as the correct ones vary somewhat with the changes in the thickness of metal, and in the proportionate amount of flange area furnished by one-eighth of the web. It will be found, though, that small variations in the length will have but little effect on the required sections, and that lengths somewhat greater than those given in the table can be used with little or no loss.

TABLE 21*f*

TABLE OF UNSUPPORTED LENGTHS OF TOP FLANGES FOR WHICH THE SAME CROSS AREA IS REQUIRED IN TENSION AND COMPRESSION

Section of Bottom Flange	Number of 1″ Holes Deducted	$\frac{l}{b}$ for Top Flange	Value of l in Feet for Top Flange	
			Top Flange Same as Bottom	Special Top Flange
2-8″×8″ ∟s 18″ cover-plates	2 out of each plate and ∟	8.5	12.8	8″ ∟s-11.6
2-8″×8″ ∟s	2 out of each ∟	9.0	12.3	....
2-8″×8″ ∟s	1 out of each ∟	4.7	6.4	....
2-8″×6″ ∟s 18″ cover-plates	2 out of each plate and ∟	8.9	13.4	8″ ∟s-12.1
2-8″×6″ ∟s	2 out of each ∟	10.5	14.3-10.8	....
2-8″×6″ ∟s	1 out of each ∟	5.3	....	....
2-6″×6″ ∟s 14″ cover-plates	2 out of each plate and ∟	11.0	12.8	6″ ∟s-11.4
2-6″×6″ ∟s	2 out of each ∟	12.4	12.8	....
2-6″×6″ ∟s	1 out of each ∟	6.2	6.4	....
2-6″×4″ ∟s	1 out of each ∟	7.5	7.7-5.2	....
2-6″×3½″ ∟s	1 out of each ∟	8.0	8.2-4.9	....
2-5″×3½″ ∟s	1 out of each ∟	8.3	7.2-5.0	....
2-5″×3″ ∟s	1 out of each ∟	8.7	7.5-4.6	....
2-4″×4″ ∟s	1 out of each ∟	8.2	5.7	....
2-4″×3½″ ∟s	1 out of each ∟	9.0	6.6-5.8	....
2-4″×3″ ∟s	1 out of each ∟	10.2	7.1-5.4	....
2-3½″×3½″ ∟s	1 out of each ∟	10.2	6.2	....
2-3½″×3″ ∟s	1 out of each ∟	11.1	6.8-5.9	....
2-3″×3″ ∟s	1 out of each ∟	12.0	6.4	....

In Table 21*f* the values for flanges with angles and cover-plates assume six-tenths (0.6) of the flange area to be in the cover-plates. The ratios for the same angles without cover-plates also appear in the tables; and for varying proportions of cover-plates the ratios can be found by interpolation. For angles smaller than 6″ × 6″ which have two holes out of each angle, either with or without cover-plates, the ratio will exceed twelve; and, therefore, no values are given in the table.

In the design of a flange for a girder, the maximum bending moment is first figured. The depth of the girder is then determined in the manner previously explained. The type of flange to be used is next decided upon. Then the positions of the centres of gravity of the flanges must be found. For flanges of two angles only, these can be taken from the manufacturers' handbooks or from Table 21*b*. For flanges composed of two angles and one or more cover-plates, the values can be taken from Table 21*d*. For four-angle flanges or for flanges with side plates, the distances will have to be calculated. In case the centre of gravity lies beyond the backs of the angles of the flanges, it should be assumed to be at that point instead. After the centres of gravity of the flanges have been determined, the effective depth d (the distance between centres of gravity of the flanges) is figured, and then the maximum flange stress is computed by the formula,

$$S = \frac{M}{d}. \qquad \text{[Eq. 2]}$$

Then the required areas of the tension and the compression flanges are calculated by the formulæ:

$$A_t = \frac{S}{f_t} - \frac{1}{8}A_w, \qquad \text{[Eq. 3]}$$

$$A_c = \frac{S}{f_c} - \frac{1}{8}A_w, \qquad \text{[Eq. 4]}$$

in which A_t = required net area of tension flange,
A_c = required gross area of compression flange,
A_w = gross area of web,
f_t = permissible tensile unit stress,
f_c = permissible compressive unit stress.

It is assumed, of course, that the points at which the top flange is to be stayed have already been determined, as discussed previously.

The two flanges are now to be sectioned, taking into account all of the factors hereinbefore noted.

The rivet pitch required at the ends of the girder must be determined before the make-up of flanges is finally settled, as it may influence the choice of the section. The principal function of the rivets connecting the flange to the web at any point is that of developing the flange increment at that particular point, which increment always acts in a direction parallel to the flange. They are frequently called upon to perform other duties, such as transferring to the web vertical loads resting on the top flange, helping to develop the bending strength of the web at a splice, and transferring to the web the radial component of the stress in bent or curved flanges.

For girders with parallel flanges, the flange stress increment ΔS per lineal inch is given by the formula,

$$\Delta S = \frac{V}{d}, \qquad \text{[Eq. 5]}$$

where V = shear at the section,
and d = effective depth at the section in inches.

The required rivet pitch p, in inches, is then given by the expression,

$$p = \frac{r}{\Delta S} = \frac{rd}{V}, \qquad \text{[Eq. 6]}$$

where r = the strength of one rivet (usually in bearing on the web).

For girders with flanges not parallel, the flange increment depends on the moment M at the section as well as upon the shear V. For a girder with V and M acting in the directions shown in Fig. 21*l*, and with flanges inclined as therein indicated, the values of the flange increments may be derived in the following manner:

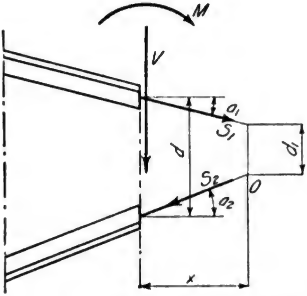


FIG. 21*l*. Girder with Inclined Flanges.

Let M = moment at section,
V = shear at section,
a_1 = tangent of inclination of top flange,
a_2 = tangent of inclination of bottom flange,
d_1 = depth in inches at point O, distant x to right,
d = depth in inches at section,
S_1 = stress in top flange,
S_2 = stress in bottom flange.

Then we have

$$d = d_1 + (a_1 + a_2)x, \qquad \text{[Eq. 7]}$$

$$S_1 = \frac{M\sqrt{1+a_1^2}}{d}, \qquad \text{[Eq. 8]}$$

and

$$S_2 = \frac{M\sqrt{1+a_2^2}}{d}. \qquad \text{[Eq. 9]}$$

Differentiating with respect to x, we have

$$\frac{d(d)}{dx} = a_1 + a_2, \qquad \text{[Eq. 10]}$$

$$\frac{dS_1}{dx} = \sqrt{1+a_1^2}\left(\frac{d\frac{dM}{dx} - M\frac{d(d)}{dx}}{d^2}\right)$$

$$= \frac{V\sqrt{1+a_1^2} - S_1(a_1+a_2)}{d}, \qquad \text{[Eq. 11]}$$

and

$$\frac{dS_2}{dx} = \frac{V\sqrt{1+a_2^2} - S_2(a_1+a_2)}{d}. \qquad \text{[Eq. 12]}$$

These equations give the values of the flange increments per inch in the direction of x. The values ΔS_1 and ΔS_2 per inch along the flanges are given by the expressions,

$$\Delta S_1 = \frac{V - S_1\frac{a_1+a_2}{\sqrt{1+a^2_1}}}{d}, \qquad \text{[Eq. 13]}$$

and

$$\Delta S_2 = \frac{V - S_2\frac{a_1+a_2}{\sqrt{1+a_2^2}}}{d}. \qquad \text{[Eq. 14]}$$

The pitch p_{tx} required in the top flange in a direction parallel to x is then

$$p_{tx} = \frac{rd}{V\sqrt{1+a_1^2} - S_1(a_1+a_2)}, \qquad \text{[Eq. 15]}$$

while for the bottom flange it is

$$p_{bx} = \frac{rd}{V\sqrt{1+a_2^2} - S_2(a_1+a_2)}. \qquad \text{[Eq. 16]}$$

The pitches measured along the flanges are

$$p_t = \frac{rd}{V - S_1\frac{a_1+a_2}{\sqrt{1+a_1^2}}}, \qquad \text{[Eq. 17]}$$

and

$$p_b = \frac{rd}{V - S_2\frac{a_1+a_2}{\sqrt{1+a_2^2}}}. \qquad \text{[Eq. 18]}$$

In case the top flange is horizontal (which is usually true) $a_1 = 0$, and the above equations then take the form,

$$p_{tx} = p_t = \frac{rd}{V - S_1 a_2}, \qquad \text{[Eq. 19]}$$

$$p_{bx} = \frac{rd}{V\sqrt{1 + a_2^2} - S_2 a_2}, \qquad \text{[Eq. 20]}$$

and

$$p_b = \frac{rd}{V - S_2 \dfrac{a_2}{\sqrt{1 + a_2^2}}}. \qquad \text{[Eq. 21]}$$

The actual shear on the web, V_w, at the section is

$$V_w = V - S_1 \frac{a_1}{\sqrt{1 + a_1^2}} - S_2 \frac{a_2}{\sqrt{1 + a_2^2}} = V - \frac{M}{d}(a_1 + a_2). \qquad \text{[Eq. 22]}$$

If the top flange be parallel to x, this equation becomes

$$V_w = V - S_2 \frac{a_2}{\sqrt{1 + a_2^2}} = V - \frac{M a_2}{d}. \qquad \text{[Eq. 23]}$$

One special case should be noted, viz., that in which there is only one force to the right of the section, which is therefore equal to V. In this case the location of the force should be chosen as the point O, and we then have

$$M = Vx, \qquad \text{[Eq. 24]}$$

$$S_1 = \frac{Vx\sqrt{1 + a_1^2}}{d}, \qquad \text{[Eq. 25]}$$

and

$$S_2 = \frac{Vx\sqrt{1 + a_2^2}}{d}. \qquad \text{[Eq. 26]}$$

The values of the rivet pitches then become

$$p_{tx} = \frac{rd}{V\sqrt{1+a_1^2} - \dfrac{Vx\sqrt{1+a_1^2}}{d}\left(a_1 + a_2\right)}$$

$$= \frac{rd^2}{V\sqrt{1+a_1^2}\left\{d - x(a_1+a_2)\right\}} = \frac{rd^2}{Vd_1\sqrt{1+a_1^2}}, \qquad \text{[Eq. 27]}$$

$$p_{bx} = \frac{rd^2}{Vd_1\sqrt{1 + a_2^2}}, \qquad \text{[Eq. 28]}$$

and

$$p_t = p_b = \frac{rd^2}{Vd_1}. \qquad \text{[Eq. 29]}$$

The shear on the web, V_w, is

$$V_w = V\left[1 - \frac{x}{d}\left(a_1 + a_2\right)\right]. \qquad \text{[Eq. 30]}$$

If the top flange be parallel to x, the above equations become

$$p_{tx} = p_t = p_b = \frac{rd^2}{Vd_1}, \qquad \text{[Eq. 31]}$$

$$p_{bx} = \frac{rd^2}{Vd_1\sqrt{1 + a_2^2}}, \qquad \text{[Eq. 32]}$$

and

$$V_w = V\left(1 - \frac{a_2\,x}{d}\right). \qquad \text{[Eq. 33]}$$

If the flanges meet at the point O, so that $d_1 = 0$, and $d = x\,(a_1 + a_2)$,

$$p_{tx} = p_{bx} = p_t = p_b = \infty, \qquad \text{[Eq. 34]}$$

and

$$V_w = 0. \qquad \text{[Eq. 35]}$$

In this case, therefore, the shear on the web is zero, and since the required pitch of rivets is infinite, the flange stress increment is zero.

The directions of the inclinations, shears, and moments have been chosen to correspond to those usually found in cantilever beams supported at the left of the section, these beams being the only ones for which these formulæ will often be used. They will usually have the top flange horizontal, and generally there is quite a stretch near the end which has a concentrated load at the outer end only, which explains why formulæ for those special cases are given. In case the inclination of a flange is opposite to that shown in the sketch, a will become minus. If both V and M become opposite in direction, the formulæ will still apply unchanged; but if only one of them is changed, the minus sign following the V in each denominator will become plus. If M reverses in direction, the arrows indicating the direction of S_1 and S_2 are to be reversed.

The pitch p in inches required to support a vertical load on a flange is given by the formula

$$p = \frac{r}{w} = \frac{rl}{W}, \qquad \text{[Eq. 36]}$$

in which r = strength of one rivet,
w = uniform load per lineal inch,
W = a concentrated load,
and l = length in inches over which the concentrated load is assumed to be uniformly distributed.

In general, there will be acting on any rivet the force from the flange increment, as well as that from the vertical load. If we call the pitch required for the flange increment p_h, and that for the vertical load p_v, we have, for a horizontal flange,

$$\frac{1}{p^2} = \frac{1}{p_h^2} + \frac{1}{p_v^2}, \qquad \text{[Eq. 37]}$$

where p = pitch required for both flange increment and vertical load. This may be put in the form

$$p = \frac{p_h\,p_v}{\sqrt{p_h^2 + p_v^2}}. \qquad \text{[Eq. 38]}$$

The curves in the upper part of Fig. 21*m* give values of p for various values of p_h and p_v.

When a flange is curved to a radius R, the radial force F per lineal inch is given by the formula

$$F = \frac{S}{R}, \qquad \text{[Eq. 39]}$$

where S = flange stress,
and R = radius in inches.
The pitch required is then given by the formula

$$p = \frac{rR}{S}, \qquad \text{[Eq. 40]}$$

where r = strength of a rivet.

This load should be treated in the same manner as a vertical load resting directly on the flanges.

The preceding discussion on rivet-pitches has applied specifically to the rivets connecting the angles to the web, and to those in flanges composed of two angles or of two angles and one or more cover-plates. After the angles are fully developed, the flange increment must pass entirely to the cover-plates. Generally, sufficient rivets for this purpose are employed as a matter of detail, because the rivets through the cover-plates should line up with those in the angles, thus giving two single shear rivets to one in bearing on the web. This point should not be overlooked, however. It will frequently be found possible to make the rivet spacing in the cover-plates double that in the flange-to-web connection. But at each end of each cover-plate close spacing is required by the specifications of Chapter LXXVIII.

Where side-plates are used under the angles, they will project beyond them, and hence there will be rivets through the side-plates and the web only, in addition to those that pass through the flange angles also. In the end portion of the girder all of these rivets will count in bearing on the web. After the angles and the side-plates are fully developed, all of the flange increment must go to the cover-plates, and the pitch will be governed either by the bearing of all of the rivets on the girder web, or by the double shearing strength of those passing through the angles only. The effect of a vertical load upon the flanges must be considered on the double shear value of the rivets through the angles only, or on the bearing value of all rivets acting on the web.

When a four-angle flange is used, it will be best to consider any vertical load to be carried entirely by the rivets in the upper pair of angles, while the flange increment is to be divided between the two pairs. In the end portion, the increment should be divided in proportion to the areas of the angles; but after the angles are fully developed, it should be divided equally between the two lines, as all of it goes into the side-plates. In this case, however, a part of the vertical load could be considered to travel out to the side-plates, and then through the lower rivets into the web.

In problems of the nature of these latter ones, it will frequently be found desirable to figure the flange increment and the vertical loads per lineal inch rather than the pitches, as the study of the stresses on the various lines of rivets can be analyzed more clearly in this manner. The combined effect of the horizontal and the vertical forces on any rivet can be easily determined. The lower portion of Fig. 21*n* can be used to advantage for this purpose; and by projecting upward to the top portion of this figure the pitch required for any given thickness of web-plate can be found.

It is to be noted that in all the cases above considered the flange increment has been figured as though the entire moment were carried by the flanges. This leads to errors on the side of safety in the end portion of girder, where a part of the moment is remaining in the web, but is correct after the bending strength of the web has been brought fully into play. When drawing up the details, it should be remembered that the figured rivet-pitches at the ends are a little smaller than those actually required, and that the use of pitches just a trifle larger than these values will do no harm.

If the rivet-pitch required for horizontal shear at the ends of a girder is known, the pitch required for the horizontal shear at any other point at which the shear has been figured can be found therefrom very quickly by the rule that the rivet-pitches will vary inversely as the shears, provided that the depth of the girder is unchanged. For the quick computation of the pitches throughout the flanges of railway girders without panels, the diagram in the lower portion of Fig. 21*m* has been prepared. In order to use this diagram, the pitches required at the ends for horizontal shear and vertical loads are first determined separately. The pitches required for horizontal shear at various points of the span are then found by means of the curves in the lower portion of the diagram. By projecting the values of the pitches for horizontal shear vertically up to the horizontal line representing the pitch required by the direct vertical load, the pitch for combined horizontal shear and vertical loads can be read directly by means of the diagonal curves. Should there be no vertical load on the flange, the pitches at the various points can, of course, be determined by means of the curves in the lower part of the diagram only.

In making the design of the girder, it is generally customary to figure the rivet-pitches only at the ends of the girders and at the ends of the cover-plates, the latter pitches being taken approximately for the purpose of figuring the net section of the flanges. Later, in the detailing, the values at a sufficient number of points are determined so as to permit of the proper detailing of the girder. Usually, the pitch should be kept constant from one stiffener to the next, and the pitches should be in even

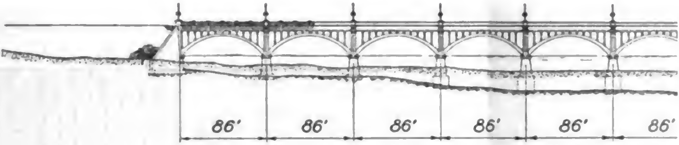


Fig. 52*m*. Proposed Thurlow Street Bri

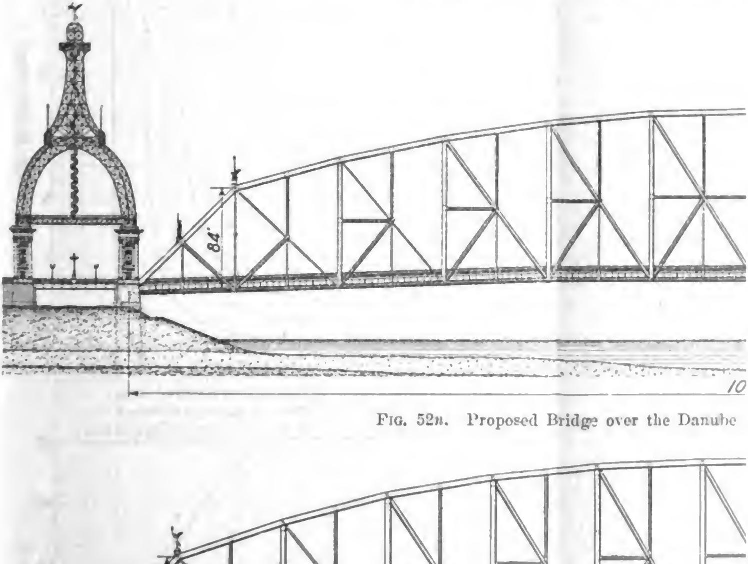


Fig. 52*n*. Proposed Bridge over the Danube

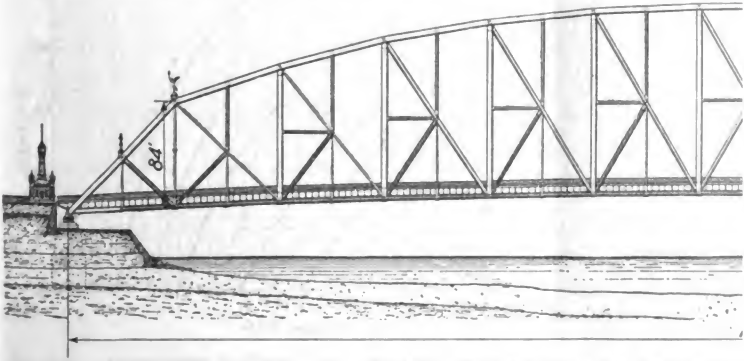


Fig. 52*o*. Proposed Bridge over the Danube

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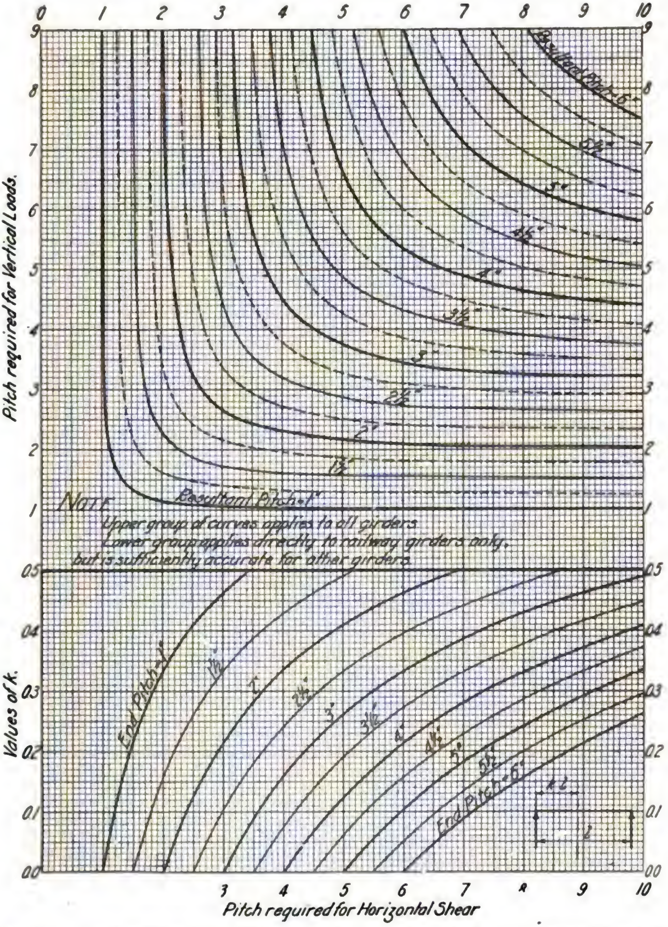


FIG. 21*m*. Rivet-pitches in Girder Flanges for Combined Shear and Direct Loads.

quarter-inches. The number of different pitches used should generally be small, even though a few rivets be wasted thereby. It will be necessary for the detailer to check the net section at the end of each cover-plate after he has detailed the flange, revising the riveting or changing the length of the cover-plate if the section is inadequate. He must also check the strength at any points where he may have had to take extra holes out of the flange.

The pitch required at web-splices will be treated later in connection with the design of the web-splice.

Occasionally the strength of a flange must be developed very quickly, as, for instance, the bottom flange of a floor-beam which has been cut back to clear a pin. In this case it will frequently be necessary to put vertical side-plates over the flange angles, extending up and catching one or more lines of rivets through the web above them. Sometimes it will be found that in such a detail $\frac{7}{8}$-inch diameter rivets spaced about $1\frac{1}{2}$-inch centres will care for the stress, so that the side-plates are not required from this standpoint. However, when $\frac{7}{8}$-inch diameter rivets in bearing on the web spaced closer than $1\frac{3}{4}$-inch centres are required, the web will be overstressed in shear. The shearing strength of a web of thickness t is 10,000 t per lineal inch, and this requires $\frac{7}{8}$-inch diameter rivets spaced $\frac{17{,}500\,t}{10{,}000\,t} =$ 1.75 inches. Thus even if closely spaced rivets will care for the stress, the reinforcing plates may be needed to strengthen the web.

After the flange section at the centre (or, more accurately, at the point of maximum moment) has been figured, and the law of variation of moment and shear determined, the next step in the design is the finding of the lengths of the cover-plates, or of the side-plates in the case of four-angle flanges. The most general way of doing this is to plot the moment diagram to any convenient scale, as previously explained, and then figure the resisting moments of the various component parts of the flange section, and plot these also on the moment diagram. For the tension flange (which will nearly always govern the lengths) the resisting moments of the various parts will usually not be constant throughout the span, as the closer rivet-pitches near the end will reduce the net sections somewhat. From 6″ to 1′ 0″ should be added at each end of the cover-plate to the length determined in this manner, and 6″ more if a railway girder is under consideration, in which the curve of moments has been assumed to be a parabola.

To illustrate the above method, suppose we wish to find the lengths of cover-plates for the girder the moment diagram of which is given in Fig. 21*d*. Assume that the web is 96″ × $\frac{3}{8}$″. The maximum moment in the girder, as scaled from the diagram, is 4,650,000 foot-pounds. The section required at this point is then calculated as follows, assuming that the top flange is supported transversely every 15 feet, and that the flange

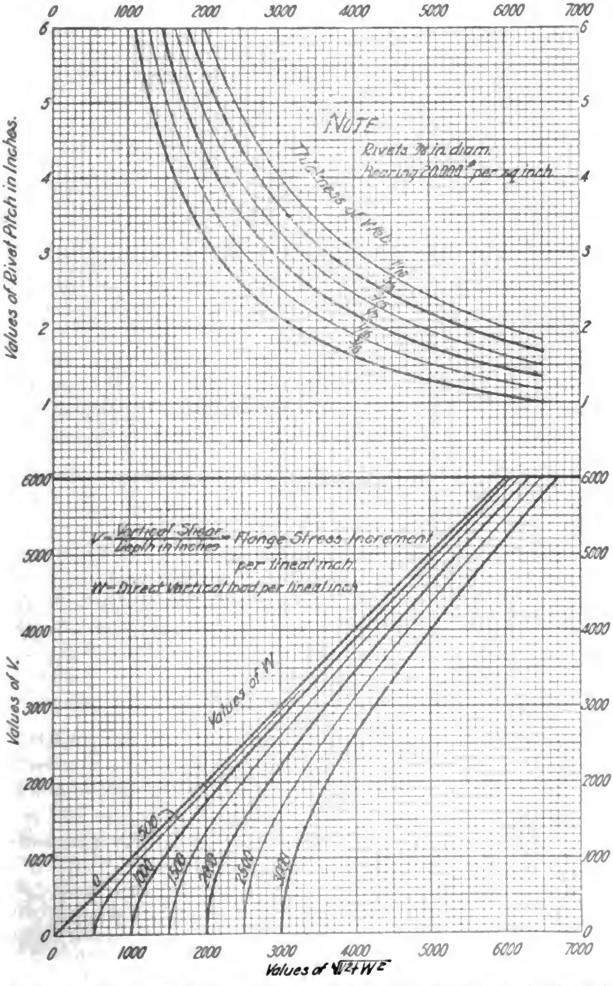


FIG. 21n. Rivet-pitches in Girder Flanges for Combined Shear and Direct Loads.

rivets are spaced 4″ centres, with one rivet only in each leg of each angle at any one section. (Assuming the gauges in the cover-plates as shown in Fig. 21*o* and those in the angles as shown in Fig. 21*p*, we need to count out but two holes per angle or plate at any section.)

$$d = 7.85'. \quad S = \frac{4{,}650{,}000}{7.85} = 593{,}000 \text{ lbs.}$$

$$S.R. \text{ net} = 37.1 - \tfrac{1}{8} \times 96 \times \tfrac{3}{8} = 32.6 \text{ sq. in.}$$

$$f_c = 16{,}000 - 200 \times 15 \div 1.5 = 14{,}000 \text{ lbs. per sq. in.}$$

$$S.R. \text{ gross} = 42.4 - 4.5 = 37.9 \text{ sq. in.}$$

Use 2Ls $8'' \times 8'' \times \frac{11}{16}'' = 21.06$ sq. in. gross, or 18.31 sq. in. net.
1 cov. pl. $18'' \times \frac{1}{2}'' = 9.00$ sq. in. gross, or 8.00 sq. in. net.
1 cov. pl. $18'' \times \frac{7}{16}'' = 7.87$ sq. in. gross, or 7.00 sq. in. net.

Total = 37.93 sq. in. gross, or 33.31 sq. in. net.

The moments of resistance of the various portions of the flange at the centre will be:

$\frac{1}{8}$ web $= 4.5 \times 7.85 \times 16{,}000 =$ 565,000 foot pounds.
Angles $= 18.31 \times 7.65 \times 16{,}000 =$ 2,240,000 foot pounds.
$\frac{1}{2}''$ cov. pl. $= 8.00 \times 8.06 \times 16{,}000 =$ 1,030,000 foot pounds.
$\frac{7}{16}''$ cov. pl. $= 7.00 \times 8.13 \times 16{,}000 =$ 910,000 foot pounds.

Total = 4,745,000 foot pounds.

Suppose that at the ends of the outer cover-plate the rivets are spaced $2\frac{3}{4}''$ centres. It is, therefore, necessary to count $1 + 0.3 + 0.78 + 0.3 = 2.38$ holes out of each angle, and $1 + 0.3 + 1 + 0.3 = 2.6$ holes out of each cover-plate, thus giving an additional net section loss in the angles of $0.38 \times 2 \times \frac{11}{16} = 0.52$ sq. in., and in the $\frac{1}{2}''$ cover-plate, of $0.6'' \times \frac{1}{2}'' = 0.30$ sq. in. The moments of resistance are, therefore, as follows:

$\frac{1}{8}$ web = 565,000 foot pounds.
Angles $= 17.79 \times 7.65 \times 16{,}000 =$ 2,180,000 foot pounds.
$\frac{1}{2}''$ cov. pl. $= 7.70 \times 8.06 \times 16{,}000 =$ 993,000 foot pounds.

Total = 3,738,000 foot pounds.

At the ends of the inner cover-plate suppose the rivet pitch to be $2\frac{1}{4}''$. It is, therefore, necessary to count $1 + 0.55 + 0.87 + 0.55 = 2.97$ holes out of each angle, thus reducing the net section of the angles $0.97 \times 2 \times \frac{11}{16}'' = 1.34$ sq. in. below that at the centre of the girder. The moments of resistance are, therefore, as follows:

$\frac{1}{8}$ web = 565,000 foot pounds.
Angles $= 16.97 \times 7.65 \times 16{,}000 =$ 2,080,000 foot pounds.

Total = 2,645,000 foot pounds.

The moment of resistance of the web should, strictly speaking, be refigured for the depth of 7.65 feet, but this refinement would not be justified.

The resisting moments of the various portions of the flange are then plotted on the moment diagram, and we find by scaling that the theoretic length of the inner cover-plate should be 49.6′, and that of the outer one 32.7′. About one foot should be added to each end of each plate, and the plates should be made symmetrical about the centre line of the girder, thus giving the final length of the inner one as 52′, and of the outer one as 35′.

It will generally save considerable time to plot, instead of the moment diagram shown in Fig. 21*c*, a similar diagram in which the vertical ordinates represent areas in square inches rather than moments. The curve of flange areas required is drawn in the same manner as was the moment diagram, the centre ordinate being the flange area required at mid-span. The areas of the various portions of the flange are then plotted, giving a figure in every way similar to Fig. 21*c*. This method ignores the decrease in the effective depth of the girder near the ends; and this should be allowed for by making the extra length added at each end of each cover-plate about one foot and six inches.

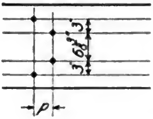


Fig. 21*o*. Layout of Rivets in Flange Plates.

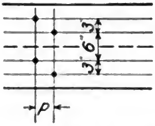


Fig. 21*p*. Layout of Rivets in Flange Angles.

For girders in which the moment curve can be assumed to be a parabola, no moment diagram need be plotted. The length of any cover-plate is then found in the following manner:

Let A = total flange area required at the centre (including ⅛ of web).
A_1 = total flange area remaining after the cover-plate in question has stopped (including ⅛ of web).
l = span length.
l_1 = cover-plate length required.

Then $$l_1 = l\sqrt{\frac{A - A_1}{A}}. \qquad \text{[Eq. 41]}$$

A and A_1 will be net sections at the points in question when figuring for the tension flange, and gross sections when figuring for the compression flange. The lengths of all of the cover-plates of the girder can be determined by a single setting of the slide rule. The quantities $A - A_1$ are first determined for all of the plates. Then the following rule is to be employed:

Opposite l on scale D set A on scale B. Then opposite any value of $A - A_1$ on scale B read the corresponding length l_1 on scale D. The scales are assumed as numbered in the ordinary manner, A, B, C, and D, from the top downward. To the lengths l_1 thus found should be added, for girders carrying uniform load only, from $1' - 0''$ to $1' - 6''$ at each end, to allow for the fact that the decrease in effective depth near the end has been ignored. In the case of a girder carrying steam railway or electric railway loading, there should be added from $1' - 6''$ to $2' - 0''$ at each end, as the moment diagram for most span lengths is a little above the parabola.

The question of the computation of stresses in continuous plate-girders was discussed briefly before. In general, their design will follow that of simple beams. The effect of stress reversal is likely to be large, and must be duly provided for. Where there is any ambiguity, or where a settlement of a support might alter materially the computed stresses, the sectioning should be liberal to provide therefor. Special end connections will frequently be necessary, in order to take care of the moments at those points.

In the design of a cantilever girder of variable depth which carries concentrated loads, the rivet pitch in any panel must be figured at the end of the panel farthest from the support, as both the depth and the flange stress are the smallest at that point.

There are two types of end details in common use. In one the girder is riveted to other steelwork by means of two end connection angles; and in the other it rests on other steelwork, or on a shoe which is carried on the masonry.

In the first type the flange angles usually run through to the ends of the girder, the connection angles extending over their vertical legs and having a tight fit against their outstanding legs. The connection angles should not be crimped, but fillers of the same thickness as the flange angles should be placed under them. Unless the girder be very light, these fillers should be three inches wider than the leg of the connection angle which rivets against them, so as to provide for a vertical row of rivets beyond.

It is generally necessary to set the connection angles to exact position, so that the girder will be of correct length, and will fit the adjoining steelwork properly. This result can be obtained in three ways. In the first method the web plate and flange angles are cut a trifle shorter than the finished length of the girder, and the connection angles are set to correct position by means of an iron frame. In the second method the web plate and flange angles are cut a little too long, and are milled to correct length after assembling. The connection angles are then set on with their backs flush with the ends of the web. The third method is similar to the second, except that the connection angles are riveted to the girder before the milling is done. The first method is the easiest, and is the one preferred by the shops. With care in the adjusting of the frames, it can be made to do very well for I-beams and shallow built beams. The second method will generally give somewhat better results, and the third is the best of the three. Whenever it is necessary that the flanges bear on adjacent steelwork, and for deep girders, either the second or the third method must be employed.

The rivets attaching the connection angles to the girder must satisfy the following conditions:

First. There must be sufficient rivets in bearing on the web, passing through the angles and the fillers, to carry the total load on the connection.

Second. There must be sufficient rivets in double shear in the angles only to carry the total load.

Third. It is preferable that there be nearly enough rivets in the angles only to carry this same load in bearing on the web, as it is evident that these rivets will be stressed more highly than those that pass through the fillers and web only.

It is desirable that the third condition be met if possible; and under any circumstances both the first and the second ones must be fulfilled. The rivets in question are almost invariably shop-driven.

The rivets connecting the end angles to the supporting steelwork must first of all be capable of carrying the total load on the connection, their single-shear value usually governing. However, these same rivets frequently carry the load from another girder as well, in which case they must also be tested for their bearing value on the supporting plate when the two girders are loaded simultaneously. For example, the rivets connecting the end angles of the longitudinal girders of a viaduct to the diaphragm web of a column must be able to carry the end shear of one girder when figured at their single-shear value, and also the maximum reaction on the column from the two girders, using in this latter case their value in bearing on the said web. Frequently the thickness of the diaphragm web has to be increased to provide for this condition of loading.

When a single line of rivets in each leg of the connection angles provides sufficient strength, the said angles should usually be 3½″ × 3½″, or sometimes 4″ × 4″ or 4″ × 3½″, in order to permit rivets in the two legs to be driven opposite without having to flatten the heads of those first driven. Where a single line fails to give sufficient rivets in one leg only, 5″ × 3½″, 6″ × 3½″, 6″ × 4″, or 7″ × 3½″ angles should be used. Where two gauge-lines in both legs are necessary, 6″ × 6″, or in some cases even 8″ × 6″ or 8″ × 8″ angles, should be employed. Occasionally wide legs are required to suit the details of the steelwork, or for other special purposes. One of these is discussed in connection with stringer details in Chapter XIX. The thickness of the angles should be such as to provide sufficient shearing strength; but this condition is rarely, if ever, in question. The best practice for railway work is to use ½ inch thickness where the ends of the girders are to be milled, and $^7/_{16}$ inch where no milling is required. For highway bridges the corresponding thicknesses are $^7/_{16}$ inch and ⅜ inch, respectively. In case rivets are used in tension (which should never be done, except in very light work), the angles must be figured for the bending moment from the rivets.

It is occasionally necessary to cut off the flanges where they meet the end connection angles, and put the latter directly against the web. While this is economical of metal, it introduces a plane of weakness just at the end of the flange angles, and also gives to the girder an unfinished appearance; hence it should be avoided whenever practicable.

Where the girder rests on a shoe or other steelwork, the end reaction is to be transferred into vertical end stiffening angles, which in turn transmit the load by bearing through the bottom flange angles to the support beneath. There are generally four of these angles at each end, although two will sometimes serve for light girders. They should extend the full depth of the girder, and should have beneath them fillers of the same thickness as the flange angles. It is best that these fillers extend three inches beyond the angles in each direction, so as to get an extra line of rivets therein on each side, unless there be a large excess of rivets through the angles. The said angles must bear very tightly on the flanges at both top and bottom, and they should fit the fillets of the flange angles as closely as possible.

The rivets connecting these end stiffening angles to the web are figured in exactly the same manner as those for the other type of end angles. The angles themselves must be calculated for the following conditions:

First. They must be able to carry the entire end reaction as a column, the unsupported length of which should be taken as one-half of the girder depth because the load varies from zero at the top to a maximum at the bottom.

Second. They must be able to carry the same load in bearing on the bottom flanges.

As the bearing on the fillet of the flange angle cannot be depended on, there is but little value obtained from the legs which are in contact with the web of the girder. It is best, therefore, to consider the area of the outstanding leg only. This allows a small amount for the value of the other leg, as a portion of this outstanding leg is over the flange fillet, and is, therefore, really ineffective. It is hardly ever necessary to make the test under the first condition. As no allowance is made for the area of the leg against the web, this should be short, and the other leg should extend nearly to the edge of the flange angle—generally to about an inch therefrom. If it should be so long as to extend past the beginning of the fillet at the tip of the flange angle, no reliance should be placed on this end portion.

The arrangement of the stiffeners will depend on the nature of the support beneath. When a pin shoe is employed, they should be in one compact group, as shown in Fig. 21q. When the girder rests on a narrow steel rocker the same detail is preferable. When the girder is carried directly on a masonry support, it is generally best to separate the two pairs, putting one pair at the end of the girder, and the other near the inner edge of the sole plate, in order better to distribute the load over the masonry. The arrangement should be as shown in Fig. 21*r*. There should be one or more lines of rivets in the fillers between the stiffening angles. Where the girder is carried on a casting without the use of a pin, either of the types mentioned can be employed. For a through railway girder, the end stiffeners depend on the end details adopted; hence they will be treated in connection with that type of girder

The proper design of the web splice is probably the most difficult feature of the detailing of a plate girder. It must evidently be so designed that the bending strength of the web is cared for in some manner; and it is preferable that the full shearing value be also developed, although it is not so essential that this latter condition be fully met when there is a considerable excess of section, as is usually the case. Evidently the ideal splice would be such that at every point the bending and shearing stresses in the web would be fully cared for by the rivets and splice plates at that exact spot. This could be obtained by the use of two plates extending the full depth of the web between the angles, and one on the vertical leg of each flange angle, each pair of plates being riveted to the web in such a manner as to care fully for both the bending and the shearing stresses in the portion of the web covered by them. This splice should, therefore, be considered the ideal splice, and the actual splice should approach it as nearly as possible. It will be found easy to make the two splice plates between the flanges care for the shearing and bending stresses in the portion they cover. It will also be simple to make the plates on the vertical legs of the flange angles care for the bending strength of the web metal under the flange angles, but it will be found impossible to consider them to care for the full shearing strength of this section without assuming the flange rivets to be considerably overstressed. It will be best, therefore, to assume the plates between the two flanges to care also for the shear in the portion of the web covered by the flanges. The only effect of this failure to care for the shear at the proper point is to cause vertical tensile stresses in the web just at the edge of the flange angles on one side of the cut, and similar compressive stresses on the other side; but the effect is small, and it is not worth while to employ extra metal or make refined calculations in order to avoid it. The plates on the vertical legs of the flange angles should then be designed only for the

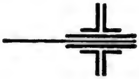


FIG. 21*q*. Arrangement of End Stiffeners for Plate-girders with Rocker Bearings.

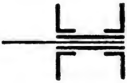


FIG. 21*r*. Arrangement of End Stiffeners for Plate-girders with Hingeless Base Plate or Pedestal Bearings.

bending strength of the web plate under the said angles. It is evident that if there should be excess flange section at the point in question, this excess can be used in taking care of the bending strength of this portion of the web, and the side-plates on the flanges can be reduced considerably, or even omitted altogether if the excess is sufficient to care fully for the bending strength of the web. In this connection it should be noted that, if one portion of the splice be made to care for the bending strength of somewhat more than the part of the web covered by it, and the other portion of the splice for the bending strength of somewhat less than the part of the web covered by it, the result will be to cause a horizontal shear in the web at the edge of the flange angles equal in amount to the excess stress which has been provided for in the first-mentioned portion of the splice. Such an arrangement also produces direct vertical stresses on the same section near the cut in the web, compressive at one flange and tensile at the other. As long as the excess amount of stress provided for in one portion of the splice is small, these effects are of little importance, particularly if the shear on the web from external loads be not large. It should further be noted that as the main vertical side-plates extend but a short distance longitudinally, while the plates on the flange usually extend considerably farther, there will be a tendency for the latter to pick up rather more of the bending strength of the web than their share, while the former plates will take up less. It will be the best practice, therefore, to provide that the splice-plates on the flange angles (or the excess area of the flange) shall care for somewhat more than the bending strength of the portion of the web covered by the flange angles. It must also be remembered that nearly all of this bending strength of the web will be developed quite close to the splice, hence the flange rivets in this region must be adequate to care for this stress.

It must not be forgotten that the bending stresses in the splice-plates vary from zero at the neutral axis to a maximum at the edge of the girder. Thus if the unit stress at the flange be 14,000 pounds per square inch, that at a point six-sevenths of this distance from the neutral axis could not be more than 12,000 pounds. The horizontal stresses on the rivets will also probably vary in some such proportion, so that the horizontal component of the stresses in the top and bottom rivets in the vertical plates should be assumed somewhat less than the stresses in the flange rivets. This result would be secured anyway, as the latter rivets have stresses from vertical shear to carry, and the horizontal components of stresses in them would have to be kept down for that reason.

Several types of splices other than the type suggested above are in common use. Some of these will now be discussed; and both their strong and their weak points will be noted.

Frequently girders are designed so that the flanges care for all bending stresses. The web splice is then usually designed solely for shear, and consists merely of two vertical plates between the flanges, with rivets proportioned for the shear only. The chief weakness of this splice is that these plates will actually pick up a large amount of the bending strength of the web under them, so that the rivets near the top and the bottom may be seriously overstressed. There may also be rather high shearing and direct stresses in the web along the edges of the flanges, as the flange section may pick up much more than the bending strength of the web covered by the flanges. The flange rivets are not usually proportioned to take up this stress quickly, so that they are frequently overstressed. These effects will be particularly severe near the end of a girder, where the shear on the web and the stresses in the flange rivets are usually already high.

Another type consists of two long strap-plates riveted to the web adjacent to each flange, and two vertical side-plates between the pairs of strap-plates. The strap-plates are figured for the full bending strength of the web, and the vertical plates for the shear. As noted previously, the unit stress in these plates will have to be less than that in the flanges, as they are closer to the neutral axis of the girder. This splice is fairly satisfactory for deep girders, but for shallow ones the vertical plates are not deep enough to serve as an efficient shear splice. There are some defects, though, when it is used in deeper girders. The vertical side-plates will actually pick up some of the bending strength of the web, and the strap-plates will pick up part of the shear, thus possibly overstressing the rivets in each case. As but a small amount of shearing strength is provided in a 12 to 18 inch space at both top and bottom of web, the tensile and compressive stresses in the said web from this effect may be rather high at the sections between the strap-plates and the vertical side-plates. Furthermore, since the strap-plates carry nearly all of the bending strength of the web, there will be shearing stresses along the planes just above and below these straps, and also direct vertical stresses along these same planes. These stresses, from shear and moment action, will be unimportant near the centre of a girder but may be rather high near the ends, where the shear from external loads is likely to be large.

Still another type of splice is made by the use of strap-plates on the flange angles to care for the entire bending strength of the web, and vertical side-plates between the flanges to care for the entire shear. The side-plates will, of course, pick up some of the moment, and the rivets near the top and bottom may be considerably overstressed. Furthermore, transferring nearly all of the bending strength in the web up to the strap-plates causes rather high shearing stresses in the web along the edges of the flanges, and also direct vertical stresses at the same sections. It will also be found necessary to extend these plates for a considerable distance to pick up the full stress they are to carry. All of these objectionable features are comparatively unimportant near the centre of a span, but may be rather bad near the ends where the horizontal shear is high.

It was stated previously that it was essential that the full bending strength of web be developed at any splice, and preferable that the full shearing strength also be developed. However, it does not follow that both of these strengths need be developed simultaneously. For a point near the end of a span, values of moments and shears nearly or quite the maximum may occur simultaneously; but near the centre of the span these maxima will be simultaneous only when the loading consists of a single concentration. It is not desirable, though, to spend much time in figuring the maximum stresses in a splice due to various combinations of moments and shears. It will be sufficient to design it so as to develop:

First. The maximum bending strength of the web, and the maximum calculated shear on the section.

Second. The maximum shearing strength of the web.

The first condition will give correct results for a splice near the end of a girder, and a variable amount of excess strength in a splice near the centre of a girder; while the second condition assures that an efficient shear splice will be used at points where the figured shear happens to be low. It will not usually be necessary to compute the splice for this second condition, unless the girder be shallow and the shear on the section small.

In working up the design of a splice of the type which was first discussed and shown to be the best, the bending strength of the portion of the web covered by the flange should first be figured, employing for this purpose the gross area of the web and the unit stress used on the compression flange for the intensity at the edge of the web. This computed strength should, preferably, be arbitrarily increased a little, as noted previously. The rivets required to transmit this stress from the web to the flange or strap-plates should now be figured. On the side next to the support they will have to care simultaneously for this stress and for that due to horizontal shear, both of which act in the same direction, and at the same time for any vertical load which may be resting directly on the flange. A close pitch on this side will usually be required. On the other side of the splice the stress due to the development of web section and that due to horizontal shear will act in opposite directions; hence at this point we should design for the minimum rather than the maximum shear at the section. An approximate value of the minimum shear, when the moment is still near a maximum, can be figured if desired; but the simplest way will be to assume that it is zero. While this will give results on the safe side, it will usually be found that the pitch required will rarely be less than that which would be used at the section if there were no splice there, consequently there is generally no object in trying to find the more exact figure. The required areas of the two flanges at the splice are now to be computed, assuming one-eighth of the section of the web effective as flange area, and also the gross area of the compression flange and the net area of the tension flange. Care must be exercised to see that no greater value is assigned to any cover-plate stopping a short distance beyond the splice than can be developed by the rivets through it located between its end and the splice. The excess areas of the two flanges are now to be determined, and also the section needed to care for the bending strength of the web under the flanges, which strength was previously figured. If the excess area of each flange provide enough section for this, no splice-plates are required; but if not, strap-plates should be put on the vertical legs of the flange angles. Plates three-eighths of an inch thick will be found to serve for any but very thick webs. If there should be no excess area in the flange, these plates will have to extend far enough to take in all of the rivets required for the development of the web; but if there is some excess area in the flange, they need not engage all of these rivets. It would not be advisable, however, to make them very short, two feet six inches being suggested as a minimum length.

The main vertical splice-plates are now to be designed so as to develop the bending strength of the portion of the web covered by them, and at the same time to carry the maximum shear on the section. The gross section of the web should be used in figuring its bending strength, the unit stress being taken as zero at the neutral axis, and equal to the unit compressive stress in the compression flange at the edge of the web. The intensity at the top or bottom of the splice-plate is then to be figured, and on dividing this value by the ratio of the sum of the thicknesses of the two splice-plates to that of the web-plate, the unit compressive stress on the gross section of the splice-plates is found. The unit tensile stress in the splice-plates will be obtained by multiplying the intensity on the gross section just determined by the ratio of the gross area to the net. When rivets at three-inch pitch are used, the unit tensile stress will be one-and-one-half times the unit compressive stress. Next is to be figured the unit shearing stress on the gross section of the assumed splice-plates, and then the unit stresses for combined shear and tension are to be computed. It will be found that in practically every case two splice-plates, each three-eighths of an inch thick, will suffice, except for shallow webs one-half inch or more in thickness, with splices located at points where the unit shear on the web is high.

The rivets in the main vertical splice-plates are now to be computed. The group on one side of the splice must be figured for the three following effects:

1. Developing the bending strength of the web.
2. Carrying the vertical shear, as a direct load.
3. Transferring the shear from the centre line of rivet group to centre line of splice.

Evidently the rivets in the top and bottom rows will be most highly stressed. The loads thereon can be figured most easily by considering the group to be acted upon by a direct load P with an eccentricity of $\frac{M}{P}$,

P being the shear, and M the sum of the moments from the first and third effects, and then applying the method given in Chapter XVI for the calculation of stresses in rivet groups subjected to bending and direct stress. The moments from the first and third effects will act in the same direction on one side of the splice, and in the opposite direction on the other side. The worst side should be figured and both sides made alike. For ordinary girders, it will be found necessary to use three lines of rivets on each side of the splice, with the spacing about three inches on centres in each row. For very shallow girders having a high shear on the section frequently more than three rows of rivets will be required.

After the splice has been designed as above, it will be necessary, especially in shallow girders in which the actual unit shearing stress on the web is not high, to investigate in order to see if the splice can develop the full shearing strength of the web. The splice-plates should be figured for the shear only; while the rivet group on one side of the splice must be able to carry both the shear and the moment due to its transference from the centre of the group to the centre of the splice.

It has been noted in the previous discussion that in figuring the bending strength of the web the unit stress at the edge thereof should be taken equal to that used in the design of the compression flange. This is sufficiently exact for ordinary cases; but it should be noted that the strictly correct way is to employ the actual unit stress at the section spliced. Taking this fact into consideration will frequently effect a legitimate saving in a splice at a point where the flange section is largely in excess of that required for moment. Care must be taken, however, not to assume the area of the flange to be any greater than can be developed by the rivets in it between the splice and the end of the girder, not forgetting that these rivets will also have to develop a portion of the strength of the web under the flange angles. The fact that the horizontal unit stress in the splice-plates at any point cannot exceed that in the web between them must also be remembered.

It is standard practice to put intermediate stiffeners at each splice, the back of the stiffener angle being at the cut in the web. This detail should be used whenever practicable.

The shoes of a plate-girder span must be designed to transmit properly the loads from the girder to the masonry or other support, to provide for expansion and contraction, and to anchor the span effectively against displacement. The limiting length of spans with sliding expansion and no rockers has for years been gradually diminishing as the live loads have increased. Until lately the author has set the limit at sixty (60) feet, but now he is inclined to place it at fifty (50) feet, especially for very heavy loads and comparatively shallow girders. From a theoretical standpoint every span should have a rocker bearing so as to make the pressure on the masonry uniformly distributed; but practically a certain amount of inequality of distribution is not harmful; hence, within reasonable limits, rocker ends for plate-girder spans may be omitted. Usually rocker ends and roller bearings go together, but a combination of rockers and sliding ends is practicable.

When spans are very short, say under twenty-five (25) feet, it is legitimate to provide no expansion whatsoever, but either to bolt both ends solidly to the masonry or to encase them in the concrete of the abutments. So doing will add materially to the rigidity of the construction.

Shoes for girder-spans resting on masonry may be conveniently classified as follows:

1. Masonry plates.
2. Cast shoes without rockers or rollers.
3. Rocker shoes without rollers.
4. Rocker shoes with rollers.

The end bearings for girders carried on other steelwork are usually of one of the two following kinds:

5. Plain sliding bearings.
6. Rocker sliding bearings.

The six types will now be discussed in detail.

Masonry plates should be used for short spans only, as they are ill-adapted to the distribution of loads of any magnitude over the masonry. They are further objectionable in that dirt collects about the bearings in sufficient amounts to rust both the plate and the girder-flange. Ordinarily there is a thick base-plate bolted to the masonry, on which bears a sole-plate that is riveted to the bottom flange of the girder. The top surface of the base-plate and the bottom surface of the sole-plate should be planed, the cut of the tool being parallel to the direction of sliding, or else both plates should be straightened. The girders should be bolted down by fox-bolts at least one and one-quarter inches in diameter, extending fully twelve inches into the masonry, the holes in the upper plate being slotted at the expansion end of the span. There should be two such bolts per bearing, arranged, if possible, so that the holes in the masonry can be drilled after the girder and its bracing are in place. The area of the base-plate must be sufficient to distribute the total load over the masonry without exceeding the specified unit pressures. The bending strength of the plate should be tested at a section along the edge of the flange. In addition, the bending moment at the centre line of the girder should be computed, and the stresses produced in the plates and flange-angles thereby should be figured on the assumption that they are acting separately, not as one thick plate, the moment being divided among the various plates in proportion to the squares of their thicknesses. In figuring the moment, the load from above should be assumed as uniformly distributed over the outstanding legs of the end stiffener angles, when four of these are used; but if only two such angles are employed, the entire load must be considered as concentrated at the centre line of the web. For a short span in which there is very little expansion, it will be satisfactory to use a single plate, which should generally be riveted to the girder.

Cast shoes without rockers or rollers are to be adopted for all spans less than fifty feet long for which the use of a masonry plate is not desirable. The shoe ordinarily consists of a single casting about six inches high. The top surface of this casting is planed, and on it there rests a sole-plate which is riveted to the bottom flange of the girder. This sole-plate either should be planed on the bottom side, or else should be straightened. The fox-bolts should be of the same size and number as for masonry plates. They may extend up through the bottom flange of the girder, or they may pass through only the bottom plate of the casting. In the latter case it will be necessary to use short bolts passing through the girder-flange, the sole-plate, and the top plate of the casting. With either detail the holes in the girder-flange and in the sole-plate must be slotted at the expansion end of the girder. The area of the bottom plate of the casting must be made large enough to distribute the load over the masonry without exceeding the allowable unit pressure thereon. It will be best to locate one rib longitudinally under the girder-web, and a cross-rib under each pair of end stiffeners. Additional cross-ribs can be used if the thickness of the bottom plate can be materially reduced thereby. The said bottom plate must be designed as a slab to transmit the load from the ribs to the masonry. In addition, the strength of the casting as a whole should be tested by figuring the stresses produced therein by the bending moment at the centre line of the girder-web.

Rocker shoes without rollers are suitable for any girder over fifty feet in length on which the load is comparatively light. The author has rarely used them, as he prefers to employ rollers, but several railroads have adopted them quite extensively, even for heavy spans. The details used have varied quite widely. In some cases pin-bearings have been employed, in others flat or cylindrical surfaces resting on cylindrical discs, and in still others flat bearing surfaces which were quite narrow in a longitudinal direction. The sliding surfaces have sometimes been steel on steel, while in other cases phosphor-bronze discs or plates have been employed in order to avoid the possibility of the sliding surfaces rusting together. For the details of shoes of this type, the reader is referred to Parts II and III of "Details of Bridge Construction," by F. W. Skinner, Esq., C.E. Special attention might be called to the Chicago, Milwaukee, and St. Paul Railway standard shoes described on p. 96 of Part III.

The author employed a shoe of the above type several years ago on a railway, deck, plate-girder span about 68 feet long. It consisted of an upper cast shoe supported on a pin, which was in turn carried by a base casting resting on the masonry. A sole-plate was riveted to the bottom flange of the girder, and rested on the upper casting. The bottom surface of the sole-plate and the top surface of the upper casting were planed, and sliding took place at this point. The base casting was fox-bolted to the masonry, and the upper casting was bolted to the girder. The bolt-holes in the girder-flange at the expansion end were slotted.

Rocker shoes with rollers should, in general, be used for all spans exceeding fifty feet in length. A great many different forms have been developed, a number of which are shown in Skinner's book just referred to. Structural shapes were formerly used quite largely in their construction, and rails have been employed to some extent also; but cast-steel shoes are decidedly better, and the price of this material is now low enough to permit of its use in all cases. Both round and segmental rollers have been employed. The latter are preferable, as the round rollers are necessarily of small diameter, and it has been found by experience that such small rollers do not work well. Furthermore, the segmental rollers are more economical except for light girders, as they generally permit the use of smaller shoes.

The details of a shoe with segmental rollers, in the pedestals of which cast steel has been used exclusively, are shown in Fig. 21*v*. The expansion shoe consists essentially of an upper casting resting on a pin which is carried on a middle casting, a roller nest, and a base casting. The fixed shoe consists of an upper casting supported by a pin which rests on the base casting. The upper castings and pins in the two shoes are identical, and the height of the base casting of the fixed shoe is equal to the combined heights of the middle casting, rollers, and base casting of the expansion shoe, so that the total height of the shoe is the same in both cases. The fixed shoe may be varied by replacing the base casting shown by a middle casting identical with that used for the expansion shoe, and a base casting having the same plan view as the base casting of the said shoe, but with its height greater by the height of the rollers. The latter type weighs more than the first form, and generally gives a larger area of base; but it can be set to correct elevation a little more easily, and may effect a saving in the pattern-making. The former kind is usually to be preferred. If in any particular case it should be satisfactory to have the fixed and the expansion shoes of different heights, the castings of the fixed shoe should be made identical with those used for the expansion shoe, the total height of the fixed shoe then being less than that of the expansion shoe by an amount equal to the height of the rollers.

The top casting consists of a top plate, a thick transverse vertical rib, and three longitudinal ribs. The upper surface is planed and the girder rests directly on it, being secured to the casting by four or eight bolts. The width of the casting is made just a trifle greater than that of the girder-flange, and the length usually about twelve inches. The distance from the top of the casting to the centre of the pin will generally be about five inches. One longitudinal rib is placed at the centre, and one at each edge. The thickness of the transverse rib is about five inches, and that of the top plate and longitudinal ribs about one or one and one-quarter inches.

The pin is made three inches in diameter, a length of one inch at the centre being increased to three and one-half inches in diameter in order to provide a shoulder that will prevent transverse displacement. The pin has a bearing throughout its length in both the upper and the middle castings, there being half pin-holes in each. In order to prevent possible uplift, bosses are cast around the half pin-holes on the two castings, and a washer slipped over them. This washer is kept in place by a thin washer against which the pin-nut bears, all as shown in Fig. 21*v*.

The middle casting consists of a bottom plate, three longitudinal ribs, and either one or three transverse ribs. Its horizontal dimensions are determined by those of the roller box, and the height is usually a little less than half of its length. The bottom plate is planed on the bottom. It must be of sufficient thickness to transfer the load from the ribs to the rollers, but rarely less than one and one-quarter inches. One transverse rib is placed under the pin. Its thickness will vary with the size of the shoe; but, preferably, it should not be less than one inch. Two other transverse ribs, usually three-quarters of an inch thick, are added when the thickness of the bottom plate can be materially reduced thereby. The longitudinal ribs are placed directly under those of the upper casting, and they should be at least one inch thick.

The rollers should generally be segmental, 6″ high by 3½″ wide, spaced 4″ centres. For light highway girders 4″ round rollers can be used, spaced 4¼″ centres. Round rollers are turned from round bars, while segmental rollers are made from forgings, the rolling surfaces only being turned. The sides must be forged straight and true, however. Round rollers are fastened to two 2½″ × ⅜″ spacing-bars by means of tap-bolts ⅞″ in diameter with flat heads ⅜″ thick, while four such spacing-bars are required for segmental rollers. In order to keep the latter kind of rollers from falling over, it is advisable to gear one roller to both the middle and the base castings by a tooth-bar at each end, which engages slots in the horizontal plates of the two castings. When this detail is used, it is necessary that the base casting be set in exact position, since an error in its location will cause the tooth-bar to tip the rollers. In order to prevent horizontal displacement, there should be two tongues ⅞″ wide and ¼″ high on the bottom plate of the middle casting and on the top plate of the base casting, engaging 1″ × ⅜″ grooves in the rollers. Both tongues and grooves should be finished. The tongues should not be located directly under the main ribs of either casting, if it can be avoided. The rollers should be protected from the weather by efficient dust-guards fastened to the middle casting and having a very small clearance with the bottom casting. Ample space must be provided inside the dust-guards for the movements of the rollers.

The base casting consists of the top and bottom plates and vertical stiffening ribs. The size of the top plate must be somewhat greater than the area enclosed by the dust-guards, so that the said guards shall never move beyond the edges thereof. The bottom plate must be large enough to distribute the load properly over the masonry. It will be found that the size required to accommodate the roller box will nearly always furnish more than enough bearing area. The top and bottom plates must be thick enough to carry the loads which come on them, but rarely less than one and one-quarter inches. The vertical ribs should usually be three-quarters of an inch thick. The use of a small number of these ribs makes the pattern-work simpler, but they should not be spaced so widely as to require unduly thick top and bottom plates. Core-holes must be provided in the vertical ribs as required, and grout-holes in the bottom plates of large shoes.

Four fox-bolts should be used per shoe. These should be at least one and one-quarter inches in diameter, and should extend fully twelve inches into the masonry. If possible, they should be located so that the holes in the masonry can be drilled after the span has been completely erected. The holes for the anchor-bolts should be about one-half of an inch larger than the bolts. In order to secure the middle casting against possible uplift, the anchor-bolts should continue up through the bottom plate thereof, the holes in this plate being slotted to allow for expansion. The nuts should not be turned down tightly, and lock nuts should be employed.

It is always desirable to be able to estimate the minimum size of base that can be used with any roller nest. With the type of shoe shown in Fig. 21*v*, this can be determined in the following manner.

Let n = number of rollers.
d = diameter of rollers in inches (height of segmental rollers).
w = width of segmental rollers in inches.
s = spacing of rollers in inches, being $d + \frac{1}{4}''$ for round rollers and $w + \frac{1}{2}''$ for segmental rollers.
L = gross length of roller in inches = net length plus 2″.
l = length of span in feet.
$l/60$ = total possible movement in inches of expansion end of span, due both to temperature and to live-load changes. We then have the dimensions given in Table 21*g*.

TABLE 21*g*
DIMENSIONS FOR ROLLER SHOES

	SEGMENTAL ROLLERS		ROUND ROLLERS	
	Transversely	Longitudinally	Transversely	Longitudinally
Inside to inside of dust-guards......	$L + 2''$	$ns + \frac{l}{60} + \frac{1}{2}''$	$L + 2''$	$ns + \frac{l}{120} + \frac{1}{2}''$
Bottom plate of middle casting...	$L + 10''$	$ns + \frac{l}{60} + \frac{1}{2}''$	$L + 10''$	$ns + \frac{l}{120} + \frac{1}{2}''$
Top and bottom plates of bottom casting..........	$L + 10''$	$ns + \frac{l}{30} + 1\frac{1}{2}''$	$L + 10''$	$ns + \frac{l}{40} + 1\frac{1}{2}''$

In designing a shoe of this type, the length and number of the rollers are first figured. The 6″ × 3½″ segmental rollers are to be adopted, except for light highway spans. The total load on the shoe is divided by the allowable pressure per lineal inch (600*d*), the number of rollers is assumed, and the required net length of one roller is calculated. To this must be added 2″ to allow for the grooves. The gross length of the rollers should be at least two inches greater than the width of the flange, as otherwise the ribs of the middle casting will be too close to the ends of the rollers. The minimum size of base that can be used is next determined from the table above, and the unit pressure on the masonry is figured. If this is less than that specified, the size of base as above found is correct; but if the unit pressure is too great, the bottom plate of the lower casting must be increased as required.

The preceding are the only figures required in the design, as the size of the base and the total height of the shoe are usually the only dimensions needed at this stage of the work. In drawing up the details, a good many other calculations are necessary. The upper shoe is to be designed so as to transfer the load from the stiffeners to the pin. The pin is to be figured for bearing. The middle casting is to be proportioned so as to transfer the load from the pin to the rollers. The bottom plate thereof is to be computed as a slab to transfer the load from the ribs to the rollers, while the bending strength of the casting as a whole must also be tested. The bottom casting is proportioned to transmit the load from the rollers to the masonry. The top plate is figured to transfer the load from the rollers to the ribs, and the bottom plate to carry it from the ribs to the masonry; while the bending strength of the casting as a whole must also be tested.

The design of the fixed shoe is comparatively simple. If the portion below the pin is to consist of two castings, no new figures are necessary, the only change required being in the height of the base casting. Occasionally, however, the top plate of the bottom casting and the bottom plate of the middle casting are reduced somewhat in thickness. The middle casting should be secured to the base casting by turned bolts, usually 15/16″ in diameter. If the portion below the pin is to be a single casting, the required area of its base is determined, and then the casting is to be designed so as to transfer the load from the pin to the masonry.

Plain sliding bearings will generally be used at the expansion ends of girders less than fifty feet long which rest on other steelwork. A plate is riveted to the bottom flanges of the girder, and bears on a plate which is riveted to the steelwork beneath. These two plates are to be straightened, or else their surfaces in contact must be planed. The sliding surface must be horizontal, so that there will be no tendency for the span as a whole to creep in one direction. The plates must be figured to transfer the load from the end stiffeners of the girder to the supporting steelwork. Provision must be made against transverse displacement of the girder; and sometimes it should be bolted down to prevent possible uplift. The details of the supporting steelwork will vary widely with the conditions of each case. Examples of this type of bearing are to be found in Figs. 19*g*, 19*h*, and 24*g*.

When a girder exceeding fifty feet in length rests on other steelwork, some form of sliding rocker bearing should be used. In one type of such a bearing a thick casting is riveted to the bottom flanges of the girder, in the bottom side of which there is planed a recess of cylindrical form, the axis of the cylinder being transverse to the girder. This recess fits down onto a narrow steel block which extends the full width of the casting, and has its top surface turned to a radius somewhat less than that used for the said casting. This block is planed on the bottom, and slides on a steel or phosphor-bronze plate which rests on the steelwork beneath. This plate either is to be planed on the top, or else it must be straightened; and its top surface must be horizontal. It must be designed to transfer the load from the steel block to the supports. As in the case of plain sliding bearings, provision should be made against horizontal displacement; and in some cases the girders should be bolted down to prevent possible uplift.

In designing expansion bearings of either of the two types last discussed, care must be taken to see that sufficient clearance is allowed for painting and field riveting after the expansion girder has been erected, and that no pocket which might collect dirt or water is formed.

A great many special types of end bearings have been designed to suit unusual conditions. The author once had occasion to support one end of a long girder on the end pin of an adjacent truss span. A number of interesting forms are to be found in Part II of "Details of Bridge Construction," by Skinner.

The shoes of girder-spans which are carried on masonry nearly always rest directly thereon; but occasionally, when the loads are very heavy, it may prove economical to rest them on grillages which are embedded in the tops of the piers or abutments. These grillages usually consist of I-beams. The top surfaces of all of the beams of a grillage must be planed at one operation after they have been assembled and riveted, in order to ensure that the shoe shall have a uniform bearing thereon; and in setting them in position every care should be taken to see that they are leveled up correctly. The bottom of the shoe which bears on the grillage must be planed.

When a shoe rests directly on the masonry, the seat therefor may be prepared in one of two ways. In the first method the top surface of the masonry is brought to correct elevation and very carefully leveled; and the bottom surface of the shoe is planed. The shoe is set on the surface thus prepared, a sheet of lead one-eighth of an inch thick often being interposed in order to allow for possible inequalities. In the second method the surface of the masonry is left about three-quarters of an inch lower than the elevation of the bottom of the shoe, no attempt being made to level it up exactly; and the bottom of the shoe is also left rough. The shoe is placed in position and properly aligned and leveled up, and the space between it and the masonry is then grouted.

The leveling up of a shoe is a rather tedious process when it is done by the use of steel shims or wedges. A much better arrangement is to place a set screw at each of the four corners. These screws should be about one inch in diameter. In light shoes they can bear directly on the masonry; but in heavy shoes they should rest on flat plates. The lower ends should be rounded slightly rather than left flat, so that the shoe will not have a tendency to shift sidewise when the screw is turned. The shoe or grillage should first be placed in correct position, and then brought to the proper elevation by means of the screws.

DESIGNING AND DETAILING OF PLATE-GIRDER AND ROLLED I-BEAM BRIDGES IN GENERAL

The method of carrying through the work of designing and detailing of bridge structures in general is treated fully in Chapter LVIII. The remainder of the present chapter will discuss special points which may arise in the design of the ordinary types of plate-girder and rolled I-beam bridges. The design of the floors and floor systems of such structures is fully covered in Chapter XIX, and that of the lateral system in Chapter XX. For the design of Trestles, Viaducts, Bridge Approaches, and Elevated Railroads, which are usually of the plate-girder type, the reader is referred to Chapters XXIII and XXIV.

ROLLED I-BEAM RAILWAY SPANS

The design of a structure of this type involves merely the floor, bracing, I-beams, and end details.

The main point to be settled in the layout of the steelwork is the number of beams per track. It will always be found that the minimum number which will carry the loading will be the most economical of metal. This will require, however, a deeper structure from grade to under clearance than will a greater number of beams, and may necessitate heavier ties or slabs. It may also in some locations cause the grade of the tracks to be raised, thus involving extra expense for both the abutments and the approach fills; and furthermore, the change in the grade may be undesirable for other reasons. All of these factors should be duly considered before the selection of the number of beams is made.

There are two kinds of I-beams on the market, the American Standard and the Bethlehem Special. As between the two, the author prefers to adopt the former in general, using the latter solely where the standard beam will carry the load only by the use of an excessive amount of metal. So long as the same number of beams per track is required in any case,

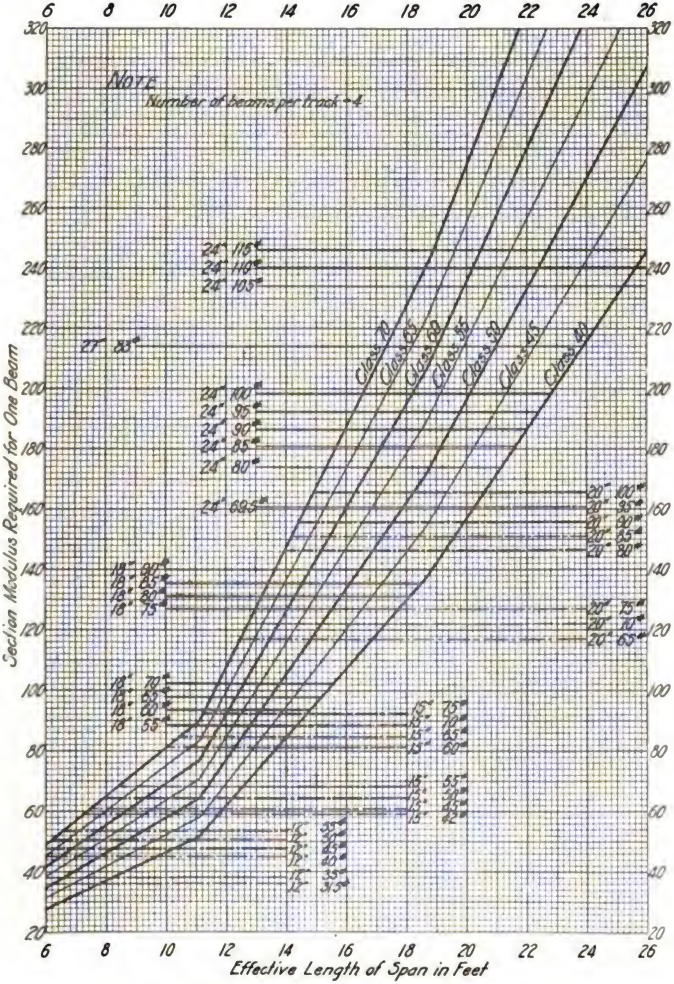


FIG. 21s. Diagram for Design of I-Beams for Railway Spans.

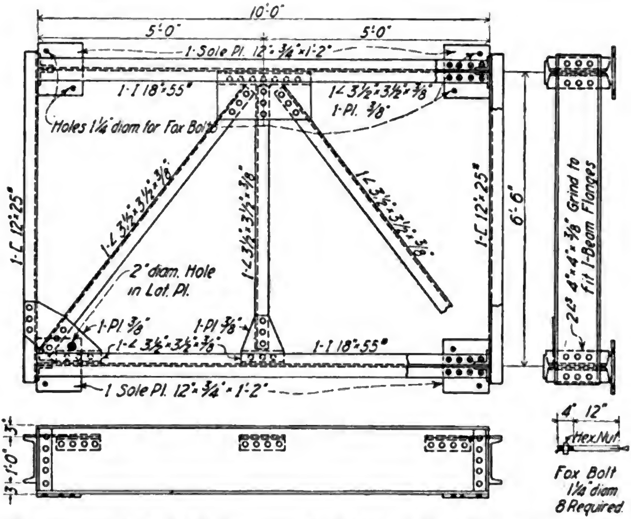


FIG. 21*t*. Details of a Railway, I-beam Span with Two Stringers per Track.

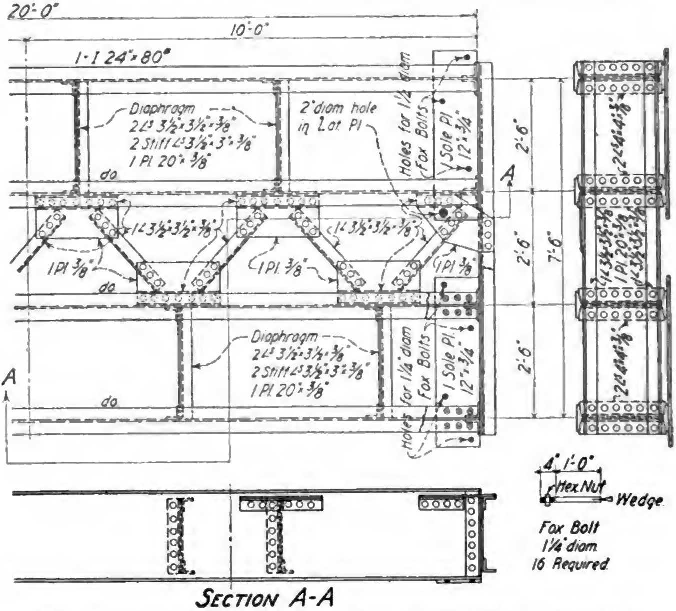


FIG. 21*u*. Details of a Railway, I-beam Span with Four Stringers per Track.

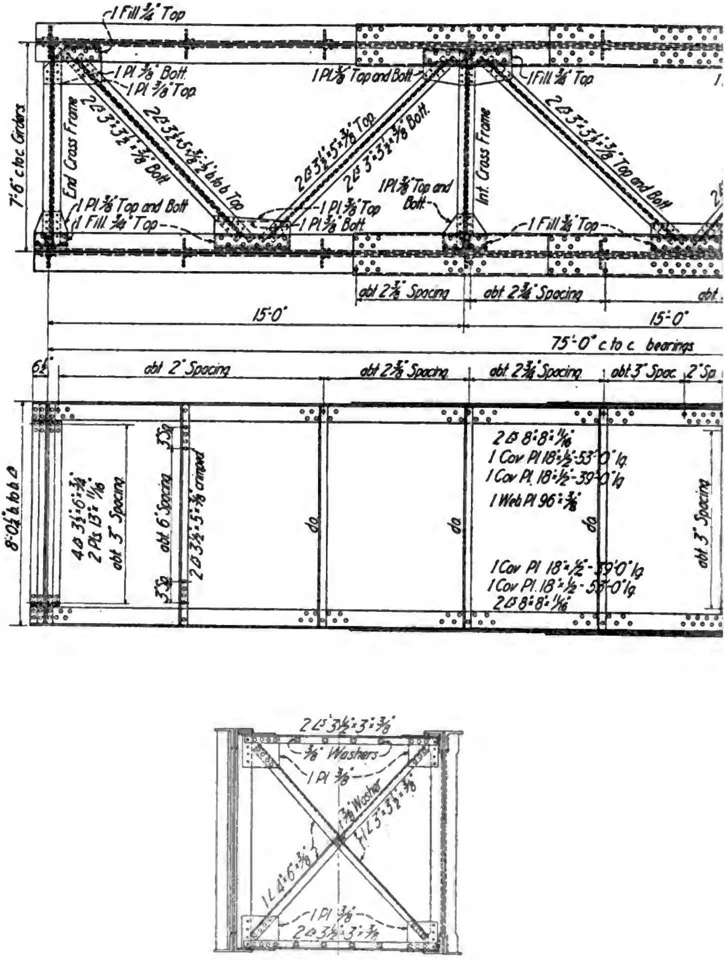
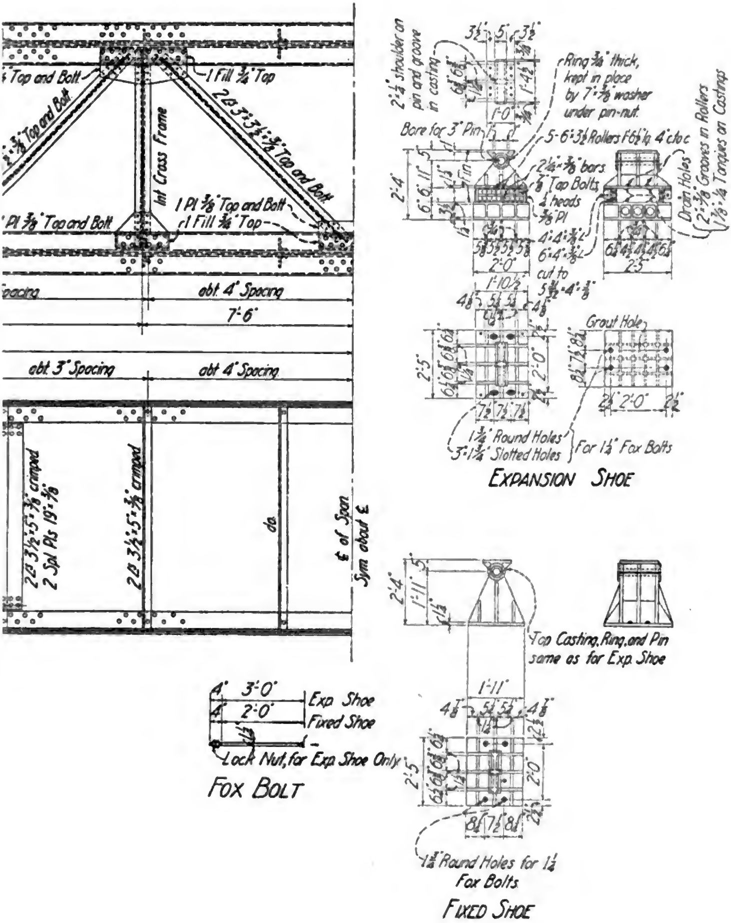


FIG. 21v. Details of a Single-track-railway, Deck, Plate-girde



Span with Flanges Composed of Two Angles and Cover-plates.

the Bethlehem beams will afford a considerable saving in weight, and a small saving in money; but where the retention of the standard beams means the employment of more lines of beams than the use of the Bethlehem sections, the difference in cost will be quite large. The author prefers the standard beam, because he considers that its strength has been better proved.

The design of the I-beams is very simple, it being necessary to figure merely the centre moment, the maximum shear on the web, and the maximum end reaction. The maximum shear on the web is not often needed with the standard beams, except occasionally when only two beams per track are used. Care should be exercised to see that the proper load per beam is taken, one-half or double the correct amount being occasionally used by mistake. The section modulus required is computed from the centre moment, and the beam is selected from the manufacturer's handbook. The calculations can be simplified by the use of Fig. 21*s*. This diagram gives directly the section modulus required per beam for various span lengths and various classes of loading, when four beams per track and open-timber decks are adopted. If any other type of floor is employed, the proportionate increase in the total load per lineal foot over that when the timber deck is used can be computed approximately, and the increase in the required section modulus quickly determined. If there should be any holes out of the bottom flange near mid-span, the net moment of inertia of the beam must be employed, as has been previously pointed out; but with proper detailing this will rarely occur.

The number of end stiffening angles required is next to be settled. If the vertical compressive stress on the web over the shoe is rather low, only two angles will be needed. These should be placed at the end of the beam so that the end diaphragm can rivet to them. If the said stress is higher (say over 10,000 lbs. per sq. in.), it will be well to place two similar angles at the other end of the shoe. These angles will rarely be required on this account unless Bethlehem beams are employed; but under heavy loads their adoption is frequently advisable in order better to distribute the load to the shoe. All end stiffeners should have a tight fit at both top and bottom.

Except for unusually long spans, say over twenty-five feet, no provision for expansion or contraction is necessary.

There are no important points in the detailing that are not fully covered elsewhere, either in this chapter or in the two preceding ones.

In Fig. 21*t* will be found complete details for a span with timber deck having two beams per track, and in Fig. 21*u* the details for a span having a similar deck, but four lines of beams per track.

ROLLED I-BEAM HIGHWAY SPANS

Usually the only portions to be designed are the floor, bracing, I-beams, and end details. Ordinarily, bracing is needed at the ends only. The number of lines of beams required will depend mainly upon the loading and the type of floor adopted. It will be economical of steel to use as few lines of beams as possible; but when a very shallow structure is called for, the employment of a larger number of beams is frequently advisable. The design of the I-beams and the end details is simple, being very similar to that required for railway spans. It should rarely, if ever, be necessary to test the shearing strength of the web of the beam. The detailing is simple, and requires no comments.

RAILWAY DECK PLATE-GIRDER SPANS

In this type of structure it is generally necessary to design the floor, lateral system, vertical sway bracing, main girders, and shoes. All of these questions have been treated quite fully already, so that hardly any comments are necessary. It might be mentioned, however, that the direct vertical load on the rivets in the top flange is to be figured by assuming the load from one wheel of the normal (not the alternative) loading to be distributed uniformly over a length equal to three times the tie spacing.

Fig. 21*v* shows the complete details for a girder span of this form in which both flanges are composed of angles and cover-plates; and Fig. 21*w* shows typical details for a span with a four-angle top flange.

RAILWAY, THROUGH, PLATE-GIRDER SPANS

In a structure of this type it is necessary to design the floor, the floor system, the laterals, the main girders, and the shoes.

No further explanations are necessary in regard to the design of the floor, floor system, and laterals. The design of the main girder itself presents a number of distinctive features. It will, of course, be materially influenced by the choice of the type of floor. When the floor system consists of transverse I-beams or troughs, the rivets in the bottom flange will frequently have to carry a part or all of the vertical load from the beams or troughs, and this fact should not be overlooked. The spacing of the intermediate stiffeners will usually have to be made to suit the floor system; and when floor-beams and stringers are employed, the said stiffeners generally carry the floor-beams. In this case they should not be crimped.

The end details of a through plate-girder are generally different from those which have previously been discussed. In the first place, a connection for an end floor-beam or brace is to be provided; and in the second place, the appearance of a girder of this type is important. On account of the latter consideration it is customary to round off the ends, bending the top flange angles through a quarter turn of two and a half feet radius or more, and running them down to the bottom flange.

The flange angles should be cut and spliced (the two at different points) a few feet from the beginning of the curve, as the bending of the long

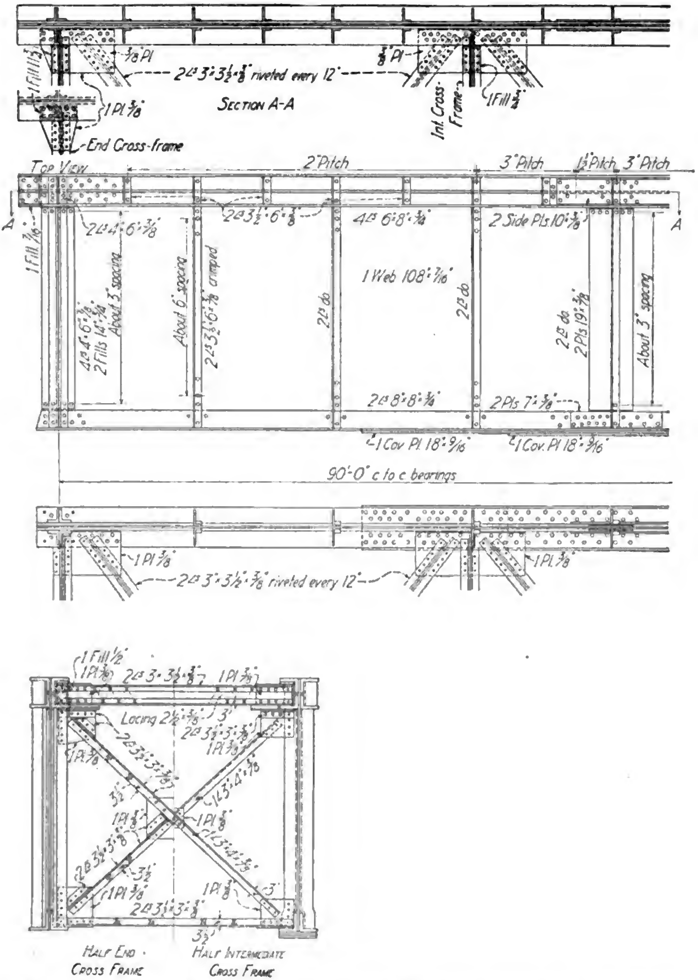


Fig. 21*w*. Details of a Single-track-railway, Deck, Plate-girder Span with a Four-angle Top Flange Section.

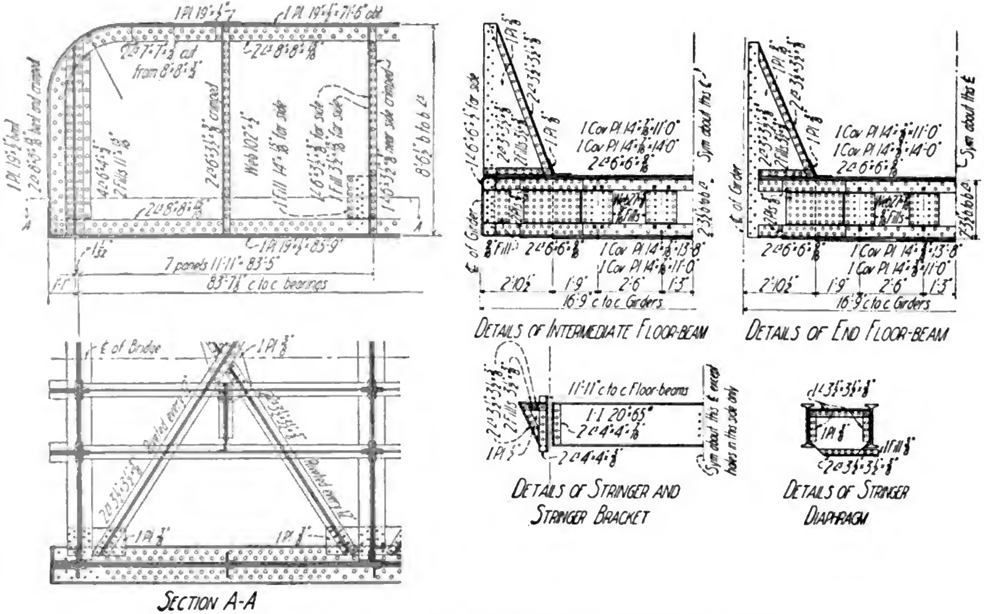


Fig. 21x. Details of a Single-track-railway, Through, Plate-girder Span.

heavy angles is a difficult shop process. The inside cover-plate should extend nearly to the curve, and should be spliced to a plate of the same width and $\frac{3}{8}''$ thick, which plate is carried around the curve and down to the bottom flange. The curved angles are usually crimped over the vertical legs of the bottom flange angles; and, therefore, they should not be relied upon to carry any vertical load. The entire end shear is ordinarily taken by four angles, arranged as shown in Fig. 21*q*. It is preferable that these angles be shop-riveted to the girder, with their outstanding legs in contact, and that the end floor-beam or bracket be field-riveted to the said outstanding legs. Occasionally one pair of the angles is field-riveted, and the floor-beam web placed between the outstanding legs of the two angles; but the bearing value of the field-riveted angles is a little uncertain, and the detail should be used only where it is impossible to take care of the floor-beam load by the other method.

When there are several successive through spans, it is best to use the rounded detail on the outer ends of the two end girders only. The end details at the other points will then be practically the same as for deck girders.

The detailing involves no points that have not already been fully treated. In Fig. 21*x* will be found the details for the end panel of an 85′ 9″ railway, through, plate-girder span.

HIGHWAY AND ELECTRIC RAILWAY, DECK, PLATE-GIRDER SPANS WITHOUT FLOOR-BEAMS AND STRINGERS

This type of structure is rarely found, except for bridges carrying electric railways only. The designing and detailing will in all essential matters follow that of railway, deck, plate-girder spans, excepting only as modified by the specifications of Chapter LXXVIII. In determining the loads for the design of the main girder, the dead load of the main girder itself may be taken from the curves of Fig. 55*ff*, after an approximate value of the total load per lineal foot has been found.

HIGHWAY AND ELECTRIC RAILWAY, DECK, PLATE-GIRDER SPANS WITH FLOOR-BEAMS AND STRINGERS

Structures of this type are very frequent, being used for most steel city bridges of short span, and for the approaches to many larger bridges. In their design there is to be taken into account the floor, the floor system, the laterals, the main girders, and their end supports. The designing and detailing embrace hardly any points that have not already been fully discussed.

HIGHWAY AND ELECTRIC RAILWAY, THROUGH, PLATE-GIRDER SPANS

The design and detailing of a span of this sort present no points that have not already been adequately treated.

CHAPTER XXII

SIMPLE TRUSS BRIDGES

WHEN plate-girders cannot be used on account of the length of span being too great, it is necessary to resort to some form of truss bridge. As a rule, for spans under one hundred (100) feet, and in many cases for spans up to one hundred and thirty (130) feet, plate-girders are adopted for the reasons given in the preceding chapter. Consequently, about one hundred (100) feet is the inferior limit for truss spans; and only in pony trusses, a type to which the author objects most vigorously, is this limit ever lowered. Various ranges in span length, more or less elastic, have been settled upon for the economical use of the different types of trusses, and these will be given in the following discussion.

For many years American bridge-designers exercised their ingenuity in devising new forms of trusses and girders, the principal object of their endeavors being to find forms involving the use of the smallest amount of metal. Each form as it appeared was tested by subjecting it to the ordeal of actual use, which showed conclusively both its merits and its defects; hence, by a process of elimination, based upon the principle of the survival of the fittest, a few forms have been retained and the others have been relegated to the history of bridge-building. As might have been anticipated, the few forms which have survived are the simplest of all; and although even at the present time one hears occasionally of some improved form of truss, the assumed improvement rarely materializes. The forms of truss that have best survived the test of time are the Pratt, Petit, Single-Intersection Triangular, Double-Intersection Triangular, and Warren. The principal ones of those that may be considered antiquated are the Fink, Bollman, Howe, Post, Lenticular, Parabolic, Lattice, Whipple, Schwedler, Kellogg, Baltimore, Radial, Pegram, "A," and Camel-Back.

The Pratt truss, Fig. 22*a*, is the type most commonly used in America for spans under two hundred and fifty (250) feet in length. Its advantages are simplicity, economy of metal, and suitability for connecting to the floor and lateral systems. Its counterbracing may be effected either by

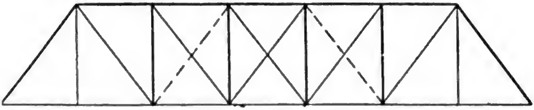


FIG. 22*a*. Pratt Truss.

the use of adjustable counter-rods, as shown by the dotted lines, or by stiffening the main members in which there is a possibility of reversion of stress. Its chords are not necessarily parallel, but may be inclined as in Fig. 22*b*. This latter form is frequently known as the Parker truss.

The Petit truss, Figs. 22*c*, 22*d*, 22*e*, and 22*f*, is a modification of the Pratt, and is generally used for spans exceeding two hundred and fifty or three hundred feet. It is comparatively simple, and, like the Pratt truss, it is economical of metal and lends itself readily to the connection of the floor and lateral systems. It has the disadvantage that its secondary stresses are rather high; but these can be materially reduced, as was shown in Chapter XI. It can be used for very long spans—even for those exceeding one thousand (1,000) feet, although no simple span as long as that has yet been constructed. The chords may be parallel, as shown in Figs. 22*c* and 22*d*; but they are generally inclined, as shown in Figs. 22*e* and 22*f*. The Petit truss with parallel chords is frequently referred to as the Baltimore truss, while when the chords are inclined it is sometimes known as the Pennsylvania truss. The sub-diagonals may run from the mid-points of the main diagonals down to the bottom chord, as in Figs. 22*c* and 22*e*, or up to the top chords, as in Figs. 22*d* and 22*f*. The former type is to be preferred, as the secondary stresses are lower, and the truss is less vibratory. It also affords a small saving of weight in riveted construction. When the panels are long, as they generally are, it is customary to support the top chords at mid-panel length by light

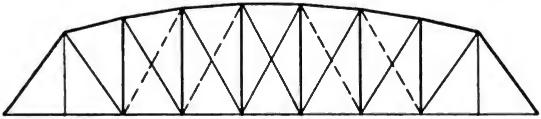


FIG. 22*b*. Pratt Truss with Polygonal Top Chord (Parker Truss).

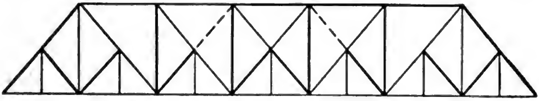


FIG. 22*c*. Petit Truss with Sub-struts and Parallel Chords (Baltimore Truss).

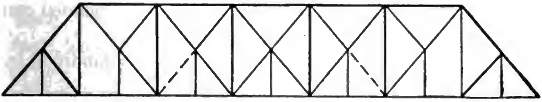


FIG. 22*d*. Petit Truss with Sub-ties and Parallel Chords (Baltimore Truss).

vertical struts extending down to the intersection of the two diagonals, as shown by the dotted lines in Figs. 22*e* and 22*f*.

The Single-Intersection Triangular truss, Fig. 22*g*, is employed for short, deck girder-spans, mainly in elevated railroad construction. It requires nearly, if not quite, as much metal as the plate-girder, and costs a little more per pound to manufacture. Almost its sole reason for existence in elevated railroads is that it obstructs the light less than the plate-girder, which type is almost universally acknowledged to be its superior in every other particular. Some people think that its appearance is preferable to that of plate-girder structures, but that is a matter of taste. It is difficult to conceive how any artistic construction can be accomplished by the employment of open-webbed, riveted deck-girders, hence it is better generally to use plate-girders wherever they are permitted. This type of truss can be built with inclined chords, but there is seldom any good reason for so doing.

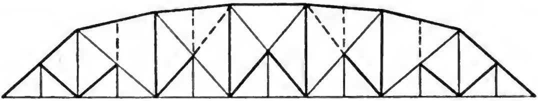


FIG. 22*e*. Petit Truss with Sub-struts and Polygonal Top Chord (Pennsylvania Truss).

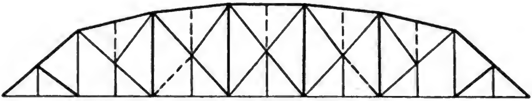


FIG. 22*f*. Petit Truss with Sub-ties and Polygonal Top Chord (Pennsylvania Truss).

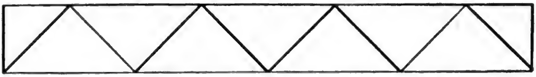


FIG. 22*g*. Single-Intersection Triangular Truss.

The form of truss shown in Fig. 22*g* is frequently modified by adding verticals at each panel-point, giving the outline illustrated in Fig. 22*h*. This type, known as the Single-Intersection Triangular truss with verticals, is extensively employed for riveted trusses of comparatively short span.

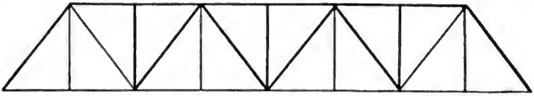


FIG. 22*h*. Single-Intersection Triangular Truss with Verticals.

It has the disadvantage that the bottom chord is subjected to rather high secondary stresses in the portions near the foot of each hanger, and the upper chord to such stresses near the top of each vertical post; but these can be pretty well eliminated, as was explained in Chapter XI.

The Double-Intersection Triangular truss, Fig. 22*i*, has managed to survive with apparently very little good reason for having done so. In addition to the unavoidable ambiguity inherent to multiple-intersection trusses, the secondary stresses in this type run high. It is claimed that it affords a certain economy of metal, but this is offset by the greater cost of the fieldwork. In some cases small vertical posts are run from the intersections of the diagonals with each other up to the middle of the long top chords so as to support them, as in Fig. 22*j*. Whether this detail is of much value is problematical, as the deflection of the intersection point must certainly cause some bending on the top chord, which bending would not exist were the vertical absent. It would appear better to deepen the chord, even at the expense of some metal. Supporting the chord at mid-panels theoretically halves the value of $\frac{l}{r}$ and thus reduces the intensity of working stress for the strut; but the gain is probably more than offset by the increased secondary stresses. In deck trusses such vertical posts are sometimes employed to support floor-beams, thus halving the panel lengths of the floor system; and in this case their use is entirely proper.

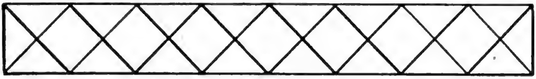


FIG. 22*i*. Double-Intersection Triangular Truss.

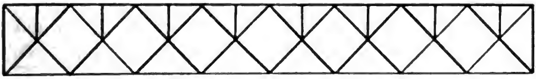


FIG. 22*j*. Double-Intersection Triangular Truss with Verticals.

The Subdivided Triangular truss, illustrated in Fig. 1*i*, has lately been resurrected by Lindenthal in his proposed bridge over the Ohio River at Sciotoville, Ohio. In the late sixties Albert Fink designed and built a truss bridge of this kind over the Ohio at Louisville, Ky., except that he used sub-ties instead of sub-struts; and in the late eighties the Illinois Central Railway Company built a bridge of this type over the same river at Cairo, Ill.—at that time the longest bridge in America. In riveted construction there is possibly a slight economy of metal in this truss over the Petit truss; but, in the author's opinion, the small saving, if such there be, is more than offset by the more sightly appearance of the latter.

The term Warren truss or Warren girder was originally applied only to the particular case of the Triangular truss in which the web triangles are all equilateral; but later writers generally use the name for any triangular truss. It is built of the single-intersection form without verticals, as in Fig. 22*k*, of the single-intersection form with verticals, as in Fig. 22*l*, and of the double-intersection form, as in Fig. 22*m*. As there is no special advantage in making the web triangles equilateral, there does not appear to be any good *raison d'être* for the use of the true Warren type.

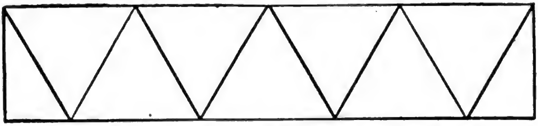


FIG. 22*k*. Single-Intersection Warren Truss.

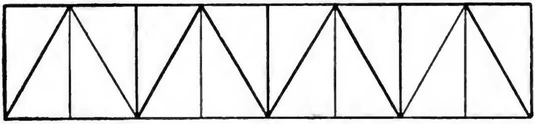


FIG. 22*l*. Single-Intersection Warren Truss with Verticals.

The Fink truss, Fig. 22*n*, and the Bollman truss, Fig. 22*o*, are so eminently lacking in rigidity that the vibrations induced in them by trains passing at high speed are truly alarming. Not only has their construction been entirely abandoned for many years, but most of the old railroad bridges of these types have been removed and replaced by better structures.

The Howe truss, Fig. 22*p*, is still used for wooden bridges on railroads of light traffic that are located far from the centres of civilization, where metal is necessarily expensive and timber is cheap; but the great weights of modern trains render it almost impracticable to design a wooden Howe truss bridge so as to withstand in a satisfactory manner the stresses induced thereby, especially in the details. The Howe truss was never

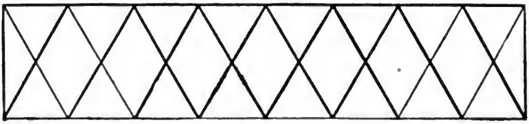


FIG. 22*m*. Double-Intersection Warren Truss.

employed to any extent for metal bridges, because its web system would manifestly be uneconomical, the diagonals being in compression and the verticals in tension. In wooden bridges of this type there is a counterstrut in each panel, as shown by the dotted lines.

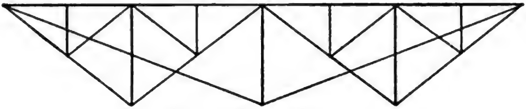


FIG. 22*n*. Fink Truss.

The Post truss, Fig. 22*q*, was quite fashionable some thirty-five or forty years ago, and was then, thanks to Col. Merrill, considered the most economical of all the existing types of trusses. Investigation has since shown that the gentleman referred to was not warranted in his conclusions, which were drawn from certain elaborate calculations based upon untenable assumptions; and nearly all the bridges that were built of this type have been replaced, the principal weak points therein being the closed columns and the loose-jointed detailing. Concerning Post truss bridges the author feels that he can speak with authority; for in 1888 he was called upon to rebuild one of the largest of these structures, which had been partially destroyed by fire. It was a very difficult piece of work to patch up the detailing so as to make it safe and passable; and it was absolutely impossible to make the bridge anything like a first-class struc-

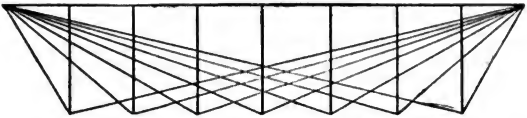


FIG. 22*o*. Bollman Truss.

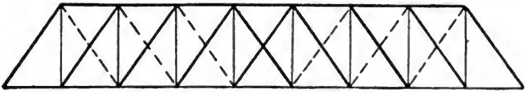


FIG. 22*p*. Howe Truss.

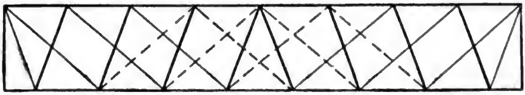


FIG. 22*q*. Post Truss.

ture, even for the light live load it had to carry. It is still standing today, but is no longer used for traffic of any kind.

The objections to the Lenticular truss, Fig. 22*r*, are its want of economy of metal, the difficulty involved in bracing the trusses laterally, and the extra expense in its manufacture due to the many varying lengths of its main members. It has been employed more in Europe than in America. To the layman its appearance may often be more pleasing than that of the ordinary American bridge; but to the initiated engineer its evident extravagance of material and shopwork is sufficient cause for its condemnation.

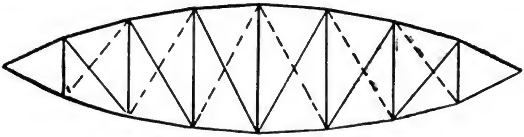


FIG. 22*r*. Lenticular Truss.

The objections to the Parabolic truss, Fig. 22*s*, are the necessity of counterbracing every panel and the impossibility of using an efficient overhead system of sway-bracing near the ends of the span.

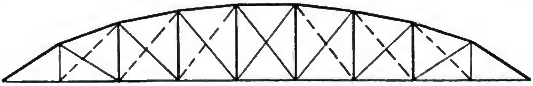


FIG. 22*s*. Parabolic Truss.

The most unsatisfactory feature of the Triangular Lattice truss, Fig. 22*t*, or any other truss involving the use of more than a single system of cancellation, is the unavoidable ambiguity in the stress distribution. There are still a few American engineers who continue to design such bridges, but their number is rapidly becoming less. They seem to think that there is some inherent virtue in several systems of triangulation, and that structures of this type are more rigid than other bridges. The general opinion of bridge engineers, however, does not endorse their views, for the great majority concede that a single system of cancellation

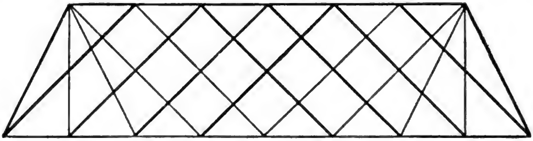


FIG. 22*t* Triangular Lattice Truss.

is the only one which is truly scientific, and that bridges built thus possess all the advantages claimed for multiple-system trusses and none of their characteristic disadvantages. The only valid plea ever made for multiple-intersection bridges is that in case of the derailment of a train on the structure, they have a better chance than single-intersection bridges of escap-

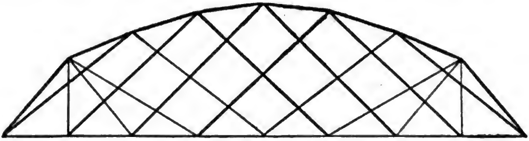


FIG. 22u. Lattice Truss with Polygonal Top Chord.

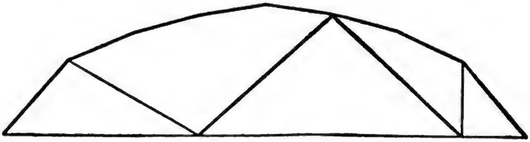


FIG. 22v.

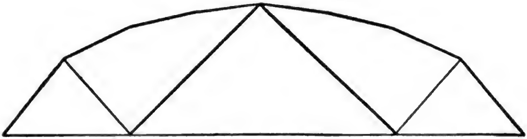


FIG. 22w.

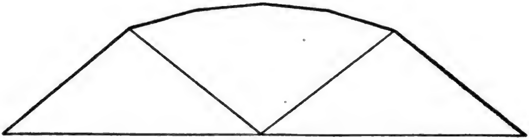


FIG. 22x.

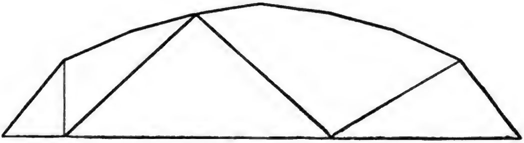


FIG. 22y.

Systems of Lattice Truss with Polygonal Top Chord.

ing total destruction; but as bridges cannot well be designed for derailed trains, this plea is not of much value. It is acknowledged by those who are posted that much more metal is required for trusses of multiple systems than for those of single-cancellation systems. The fact that the Whipple truss, Fig. 22*z*, is more economical of metal than the Pratt truss, Fig. 22*a*, after certain span lengths are exceeded, does not militate against the correctness of this statement; because for such long spans the Petit truss, Figs. 22*c*, 22*d*, 22*e*, or 22*f*, replaces the Pratt, and it is more economical of metal than the Whipple. Again, ordinary multiple-system trusses are more subject to secondary stresses than are single-cancellation trusses, partly for the reason that the various members of the latticed webs are riveted together in the field and are, in consequence, distorted by the drifting, unless the trusses are assembled and reamed in the shop, and partly on account of the fact that if one truss system happens to be more highly loaded than another, the panel-points thereof will deflect more than those of the other system.

It is common practice in Europe, but almost unheard of in America, to use polygonal chords in lattice trusses, as in Fig. 22*u*. When for the purpose of computing stresses such a truss is divided into its component systems, as in Figs. 22*v*, 22*w*, 22*x*, and 22*y*, it will be seen that the top chord stresses have to travel from panel-point to panel-point through bent members; and were not these bent struts stayed at the points of bending by the web members of the other component trusses, the structure would collapse. But the chord struts of one component truss can be stayed only by inducing rather large stresses in the other component trusses; hence the calculation of the web stresses is exceedingly intricate. Strickly speaking, they are not solvable, except by the methods discussed in Chapter XII for the calculation of stresses in indeterminate trusses.

The Whipple truss, Fig. 22*z*, is a double-intersection type, similar to the lattice, with two systems of cancellation only. It used to be very common in America, but nowadays it is seldom employed in designing.

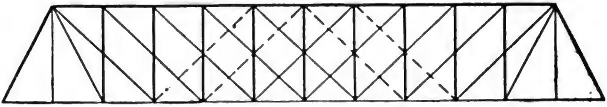


FIG. 22*z*. Whipple Truss.

The ambiguity of stress distribution that it involves when the number of panels is not exactly divisible by four (4) is well worthy of consideration.

The Schwedler truss, Fig. 22*aa*, is a variation of the Whipple that is suitable for long spans. Its appearance is against it, for not even its originator could hold that it has any claim to beauty. It might, however, show some economy of metal as compared with certain other types of trusses.

The Kellogg truss, Fig. 22*bb*, is a variation of the Pratt, but is inferior to it in rigidity. It was a freak and never had any valid claim to existence, hence its life was short.

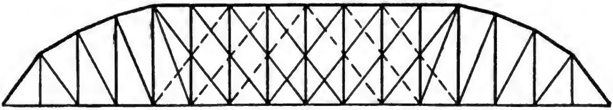


Fig. 22*aa*. Schwedler Truss.

The Pegram truss, Fig. 22*cc*, was used quite largely by its originator, George H. Pegram, Esq. C.E., but no one else seems to have made use of it. It is more sightly in appearance than the Pratt truss; but as, of necessity, it involves the use of suspended floor-beams, it is inferior to that standard type. Mr. Pegram claimed an economy of metal for his type of truss as compared with the Pratt and other trusses; but if it really existed, it was more than offset by the greater cost of shopwork. The peculiar feature of the truss is that the panel lengths of the upper chord are equal to each other, notwithstanding the polygonal outline.

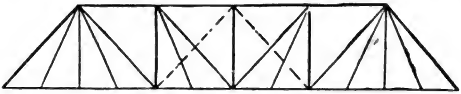


Fig. 22*bb*. Kellogg Truss.

The "A" truss bridge, Fig. 22*dd*, patented many years ago by the author, served a good purpose for some time until the modern riveted Pratt truss bridge was evolved. Quite a few of them were built, and nearly all are still in use, notwithstanding the fact that some are frequently overloaded as much as sixty (60) per cent. It is the most rigid short-span, pin-connected bridge ever built. Its appearance is odd but not displeasing.

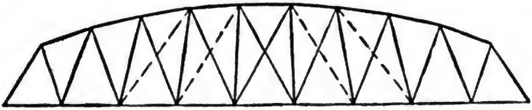


Fig. 22*cc*. Pegram Truss.

The Camel-back truss, Figs. 22*ee* and 22*ff*, is a variation of either the Pratt truss or the Petit. It was called into existence by a large bridge company for the purpose of saving some metal and the shop cost of changing the inclination of the top chord at each panel-point. In appearance the truss is uncompromisingly ugly; and the sudden change of chord inclination came very near on two occasions to proving the said bridge company's undoing, for its computers failed to note the necessity of counterbracing the panel where the change occurred. The result was the actual reversing of the stress in the panel, which, had it lasted more than an instant, would certainly have destroyed the bridge. The weak diagonals in the existing structures of this type were stiffened free of charge by the manufacturing company soon after the defect in their design was discovered, and thus accident was avoided.

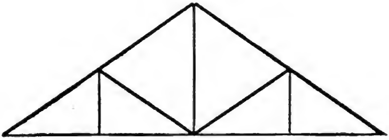


FIG. 22*dd*. Waddell's " A " Truss.

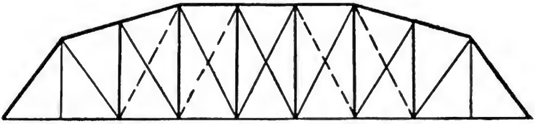


FIG. 22*ee*. Camel-back Truss.

The K-type of webbing, which is being used in the cantilever arms and the anchor arms of the new Quebec Bridge, could be adopted for long-span simple trusses as well, although it is difficult to secure a satisfactory arrangement of the members in the end panels. Figs. 22*gg* and 22*hh* suggest outlines which might be employed. The latter is the better looking of the two, but weighs some ten or fifteen per cent more. The weight of the truss shown in Fig. 22*gg* is about the same as that of a Petit Truss. The chief superiority of the K-type over the Petit is its lower secondary stresses; but its inferior appearance will probably prevent its being used to any great extent.

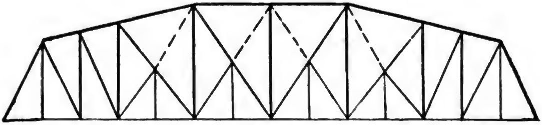


FIG. 22*ff*. Camel-back Truss with Subdivided Panels.

Various combinations of trusses and arches have from time to time been suggested, but none of them have survived, because they violate the principle that "Simplicity is one of the highest attributes of good designing."

In respect to pony-truss spans the author has maintained for years that such structures should be ruled out entirely, and that under no circumstances is it necessary to build them, because they can be replaced by plate-girder spans by the expenditure of more money. The main objection to pony-truss bridges is that no man can tell even approximately what is the ultimate strength of their wholly or partially unsupported top chords.

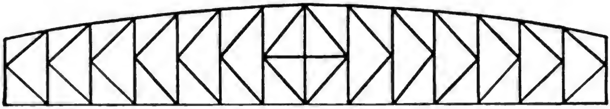


FIG. 22*gg*. K-Truss.

The length of span at which it pays to change from parallel chords to curved or, more properly speaking, polygonal chords, will vary with the class of bridge; but it is seldom advisable to adopt the latter for spans under two hundred feet. The greater the panel length the greater the limit of span for parallel chords, consequently it will generally be found shorter for highway bridges than for railway bridges. This curving of the top chords of long through spans has sometimes been carried to such excess as to approach very closely the old parabolic trusses, in which the curve extends from end-pin to end-pin. In a large and important bridge over the Mississippi River the top chords of the main spans, which exceed five hundred feet in length, are so curved as to involve the use of a very shallow portal, allowing but the ordinary clear headway beneath. Such excessive curvature causes the top chord to do most of the work of the web and makes the latter too light and vibratory. It also necessitates the use of counters or stiff main diagonals almost up to the ends of the span. A proper curvature of the chords is not only economical of both metal and money, but also is æsthetic, adding greatly to the appearance of most bridges, consequently this feature should be encouraged, but not, of course, to excess. The best curvature of chords

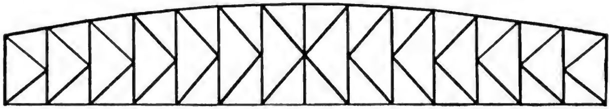


FIG. 22*hh*. K-Truss.

for any span can only be determined by experience, the controlling factor being reversion of web stresses. In general, it may be said that the greater the arching the more artistic the effect. For highway bridges it can be made greater than for railway bridges, because the effect of impact is less in the former than in the latter; nevertheless, even in highway bridges the curvature must not be carried to excess on account of the tendency of light web members to set up vibration from insignificant moving loads.

The following examples from the author's practice will give an idea of what top-chord curvature can be employed legitimately.

In a design for an eleven hundred foot highway span (shown in Fig. 52*o*), for which the live load is 4,500 pounds per lineal foot and the dead load 20,000 pounds per lineal foot, the truss depth is one hundred and fifty feet at mid-span and eighty-four feet at the main hips or second panel-points. In a similar design for a highway bridge of one thousand and forty feet span (shown in Fig. 52*n*), for which the live load is 4,000 pounds per lineal foot and the dead load 17,000 pounds per lineal foot, the truss depth at mid-span is one hundred and forty feet, and at the main hips eighty-four feet. In a design for a six hundred and twenty-four foot, double-track railway span the respective depths are ninety-three feet and sixty-five feet. For a similar span of four hundred and ninety feet they are seventy-five feet and fifty-two feet respectively. For a five hundred and sixty foot, double-track railway, motor, wagon, and pedestrian bridge the depths are ninety feet and sixty-five feet. For two five-hundred-foot, single-track, railway spans, carrying sidewalks within the trusses and wagonways outside, the centre depth was taken at seventy-two feet and the hip depth at forty-eight feet. For a similar bridge and a span of four hundred and twenty-three feet the centre depth was taken at sixty-five feet and the hip depth at forty-seven feet. For a single-track railway span of three hundred and sixty-two feet the centre depth is sixty-two feet and the hip depth forty-five feet. In this case the panels are longer than in the one directly preceding, which will account for the relatively greater truss depths in respect to span length.

In regard to the form of polygonal top chords, experience shows that an arc of a parabola from hip to hip is in every way the best layout, because the curve thereof is the most sightly.

The inferior limit of span length for which it is advisable to use subdivided panels will vary somewhat with the style of structure and with the panel length. In general it may be stated that for railway bridges this length is about two hundred and seventy-five or three hundred feet, and for highway bridges with panels not much in excess of twenty feet long it is about two hundred and twenty-five or two hundred and fifty feet. Where shallow floors are necessary, the subdivided truss can be used to advantage for much shorter spans. Economy of metal should not be the only or even the principal consideration in determining this limit; for general appearance and proper sizes of sections are more important factors. In railway bridges where the panels are made long for the sake of economy of metal, it takes a rather great truss depth to warrant one in running the main diagonals over two panels, and to prevent giving a squat appearance to the span; and the truss depth for any span is something which cannot be varied much for any such object as the use of subdivided panels; consequently, the longer the panel adopted the greater must be the limiting span length for the legitimate employment of the Petit truss. The heavier the span the greater is the permissible truss depth, especially with polygonal top chords, consequently (on account of appearance) the shorter is the span length that limits the legitimate subdivision of panels.

Again, in very light bridges there is often no weight of metal saved, even in spans of considerable length, by subdividing the panels; and as such subdivision tends to complicate the detailing and thus to increase the pound price of the metal, as well as to render the span vibratory because of unduly light truss members, it is evident that a designer should think twice before deciding to subdivide the panels in any design for a span of about the usual inferior limit for Petit trusses.

Vertical end posts for through bridges are seldom employed in modern bridge designing. There are three good reasons for this, viz., first, each vertical end post necessitates an idle panel length of bottom chord, which piece, although stiffened, has a tendency to promote vibration of the structure because it has no stress in it; second, vertical end posts involve the use of more metal for trusses than do inclined end posts; and, third, the appearance of a truss with the end posts vertical is by no means as sightly as that of a truss where they are inclined.

Until quite lately it has been the almost universal custom among American engineers to make all the panels of a bridge truss of the same length, but Hodge in his "Municipal Bridge" at St. Louis (originally known as the "Free Bridge") has departed from this practice. In *Engineering Record*, Vol. 68, page 322, Modjeski, in an article entitled "Design of Large Bridges," while admitting a possible slight economy by the varying of the panel lengths, as well as better appearance due to a more uniform inclination of web diagonals to the vertical, pronounces against the innovation on the plea that these advantages are outweighed by the gains in shopwork and in erection from uniformity of panel length. The author is inclined to agree with Mr. Hodge and to disagree with Mr. Modjeski, for the reason that the advantage in shopwork by adhering to uniform panel lengths is trifling, while the loss in æsthetics may be great. The powerful pressure of shop influence has too long had a tendency to throttle the progressive innovations of all American bridge designers; and it is just as well for the latter once in a while to assert their independence, even if by so doing they increase somewhat the cost of their structures.

Considerable attention has been given by writers on the subject of bridges to continuous girders, the result being simply a great waste of printer's ink. Few American engineers will countenance the building of continuous girder bridges, except for swing spans. The unanswerable objection to the use of structures of that type is that the correctness of the stresses, found by a most elaborate system of calculations, is dependent upon the exactness of the elevations of the bearings on the piers. The least variation of these from correctness will upset completely every stress in all the trusses that rest on the pier where the variation exists, to say nothing of how it will affect the stresses in the trusses of the other spans. Now, as all piers are compressible, and as many of them settle materially before coming to a final position, especially when timber or piling is used in their construction, it follows that in many cases no reliance can be placed upon the permanency of the bearing elevations, or, consequently, upon the correctness of stresses calculated for continuous girders. The only advantage claimed for the continuity is a saving of metal; but, setting aside the unreliability just shown, this economy would be more than offset by the risk to the whole structure from an accident to one span or to one pier. It is proper to state, though, that not all American bridge engineers agree with the author in his drastic condemnation of continuous spans; for one of the most prominent of them, Gustav Lindenthal, Esq., C.E., has lately proposed a structure of that type for a crossing of the Ohio River at Sciotoville, Ohio, as shown in Fig. 1*i*. The conditions at that crossing, however, are exceedingly favorable for continuous spans, because there is a good solid rock foundation from end to end thereof only a few feet below the low-water elevation.

There is a type of truss, mentioned incidentally in Chapter II, which the author has dubbed the "Hybrid" truss, because it is a combination of the pin-connected and the riveted types, and, like most hybrids in general, it is illegitimate. Its top and bottom chords and its verticals are stiff members that are riveted together, but the diagonals are non-adjustable eye-bars, which are attached to pins that pass through the connecting plates at points which lie on the centre lines of the diagonals but not on those of the chords. Any one who is experienced in the manufacture and erection of bridges will recognize the impossibility, with only the ordinary shop manipulations, of making the component members of any diagonal of such a structure act together so as to distribute the stress uniformly between them; because when the main members are attached to the connecting plates and are brought into position by the drift pins in advance of the final reaming of the rivet holes, they will not come to really true position, as they would were the structure pin-connected. This type of construction was used to a slight extent in the Beaver Cantilever Bridge; but in this case the shopwork was carried out in such a careful manner that the above criticism does not apply.

Trusses of this kind having all their main diagonals as well as their counters adjustable would be a good kind of construction for highway bridges, and would be a decided improvement on the light, pin-connected, vibratory structures that are encountered on the wagon roads throughout the entire extent of the United States; but as adjustable members are not proper for railway bridges, the Hybrid truss ought to be debarred therefrom.

As a rule, only two trusses per span are used in American engineering practice; and it is rare for conditions to require more. Even at the expense of a somewhat greater amount of metal, the two-truss construction should be given the preference wherever it can be employed; for the unavoidable uncertainty in the distribution of the loads makes the structure with more than two trusses objectionable. In addition to this, there is the unsatisfactory feature of the racking of the floor-beams and sway-bracing due to the fact that the various trusses deflect different amounts. Special attention must be given to their designing and detailing in order to prevent overstresses from such unbalanced loading. In short spans, though, where the truss deflections at the most are not large, this consideration may not be of any serious moment.

The employment of more than two trusses is rarely necessary unless the floor system requires it. For a wide spacing of trusses, the floor-beams become deep and heavy, which under certain conditions could not be permitted. Where this is the case, it is necessary either to use two or more spans side by side or to adopt a single span having more than two trusses. The former method gives the best arrangement for the trusses themselves, but it is uneconomical in the superstructure as a whole as well as in the substructure. Moreover, for the same floor space it requires a wider structure out to out than the latter; and often the extra width would not be permissible, especially in city bridges where the construction must be confined to the width of the street.

In highway bridges, the cantilevering of part of the floor beyond the trusses permits a shallower and more economical floor system than would be the case if the trusses were placed at the outside of the structure; and, as a rule, this construction precludes the necessity of more than two trusses. Only in the case of a closely limited depth from the floor level to the under side of the steelwork will more than two trusses be necessary, and under such a condition every effort should be made to raise the grade in order to provide sufficient space for the floor system without increasing the number of trusses.

In railway bridges the spacing of trusses, as well as the number of trusses per span, depends upon the number of tracks to be accommodated and the clearance required. Fig. 22*ii* shows the clearance diagram for single-track through bridges on tangent, while that for structures on curves is given in Figs. 8*e* and 8*f*. For multiple-track bridges the horizontal clearances shown in these diagrams have to be increased by the distances from centre to centre of tracks. No more than two trusses should ever be used for a double-track structure, and this number will be sufficient for spans with more than two tracks, except where the use of shallow floors cannot be avoided. While it is true that in certain instances the grade cannot be changed so as to increase the depth available for the floor system, it is not always the case; and the grade should not be held down, if it can be raised legitimately and a better structure be thus secured. It must also be remembered that the adding of intermediate trusses causes an increase in the spacing of the tracks adjacent to the bridge, which arrangement may be objectionable. Instead of adding a third truss in a four-track structure, two of the tracks can be supported on cantilevers, one on each side. The same is also possible in a span designed for six tracks; and if this does not give a sufficiently shallow floor, a third truss can be placed at the centre.

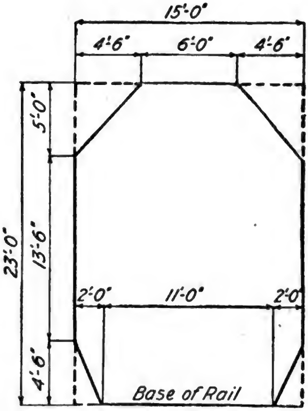


FIG. 22*ii*. Clearance Diagram for Through Bridges on Tangent.

As has been previously stated, the use of three trusses is less objectionable in short spans than in longer ones. Moreover, they often work out very satisfactorily in structures having a considerable skew. Again, the addition of a third truss has been resorted to very successfully in strengthening old bridges, thus increasing their usefulness. As a general proposition, however, more than two trusses per span should rarely be employed.

Skew-spans should be avoided if it is reasonably practicable to do so. Where it is absolutely necessary to use them, the arrangement of the panels and the outlines of the trusses require special attention. As far as possible the two trusses should be made alike, and this can be done only when the two ends have the same skew. It is, therefore, advantageous to make the skews the same, at least so far as the steelwork is concerned, even if it is not possible to do the same with the substructure. The web members of the trusses should be so arranged that the floor-beams and the sway frames will be at right angles to the truss-planes. Moreover, it is best to keep the end posts parallel so as to avoid warped portals. Skew portals in themselves are sufficiently troublesome without complicating the matter by warping them as well. It is true, though, that warped portals cannot be avoided in all instances; and when this is the case, it is necessary to make the best of the unsatisfactory condition.

The most favorable case is when the skew (or, more strictly speaking, the offset thereof in the perpendicular distance between central planes of trusses) is such (or can be so adjusted) that it can be adopted economically for the panel length of the span. This necessitates a span length integrally divisible by the length of the skew, which desideratum can generally be accomplished only by an arbitrary location of the piers. Fig. 22*jj* shows the arrangement of the trusses for this condition. In case this is not practicable, it may be possible to make the first two panels at the acute end of each truss of a length equal to that of the skew (necessitating one panel at the opposite end to be of the same length), and then divide the intervening space into panels very nearly equal to the skew. This case is practically the same as that shown in Fig. 22*jj*. For spans (mostly short ones) in which neither of the two methods just outlined can be employed, the arrangement shown in Fig. 22*kk* might work out satisfactorily. In this case each truss is divided into a number of central panels with lengths equal to that of the skew and a panel at each end equal to one-half of the difference between the total length of the span and that of the central portion comprising the other panels. When the end panels are shorter than the intermediate ones, the intermediate floor-beam nearest to each end can be made square and riveted to the skew end floor-beam, as shown in Fig. 22*kk*, or it can be skewed between the hip

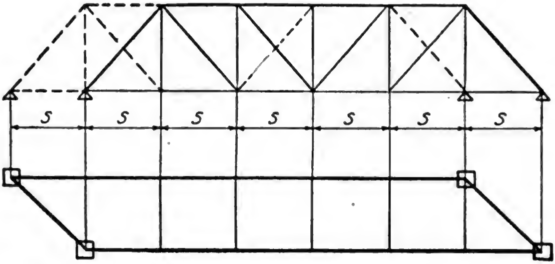


FIG. 22*jj*. Arrangement of Trusses for a Skew Span in which the Skew Can Be Adopted for the Panel Length Throughout.

vertical of one truss and the obtuse end of the other; but when the end panels are longer than the intermediate ones, the second scheme must be used.

Where the skew is considerably shorter than an economical panel length, the arrangement shown in Fig. 22*ll* will have to be resorted to;

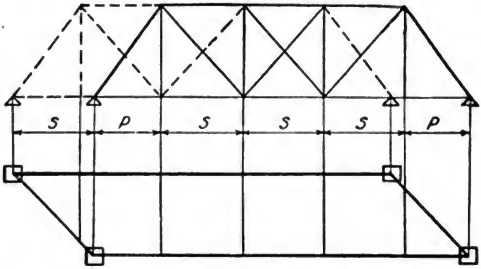


FIG. 22*kk*. Arrangement of Trusses for a Skew Span in which the Skew Cannot be Adopted for the Panel Length Throughout.

and where the skew is longer than an economical panel and still not long enough for two panels, that shown in Fig. 22*mm* must be used. Both of these require warped portals, which present difficulties in their construction. This might be avoided by adopting vertical end posts with tension end diagonals, and placing a skew portal in the vertical plane containing the said end posts. This arrangement not only requires extra

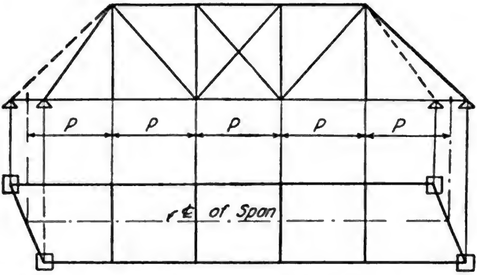


FIG. 22*ll*. Arrangement of Trusses for a Skew Span in which the Skew is Too Short for an Economical Panel Length.

metal but also produces an unæsthetic truss for through spans; and the advisability of resorting to such a peculiar layout should be given careful consideration before it is adopted. In deck spans the end diagonals should be compression members, and the end sway frames should connect between the end verticals that are needed to carry either the end floor-beams or the top chord when the live load is supported directly on the latter.

When the skew extends over two or more panels, one of the arrangements shown in Figs. 22*jj*, 22*kk*, 22*ll*, or 22*mm* can be employed by shifting the trusses the required amount with respect to each other. In any case the panels should be arranged so that the intermediate floor-beams are all square to the trusses. Moreover, skew end floor-beams should be used in practically all cases. If this requires especially heavy skew beams, or beams difficult to provide, one end of each square beam can be supported directly on the masonry and be connected to the ends of the trusses with substantial bracing frames, or the skew floor-beams can be supported on the masonry by sliding bearings. Also the lateral system must be made complete, as the entire steelwork must act as a unit in order that the design may be in accord with the most approved modern practice.

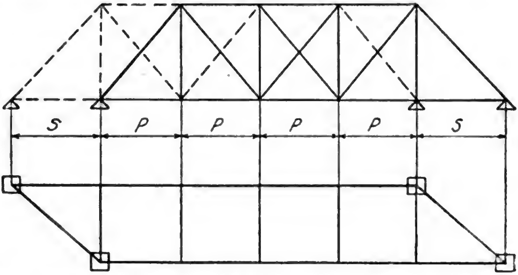


FIG. 22*mm*. Arrangement of Trusses for a Skew Span in which the Skew is Too Long for an Economical Panel Length.

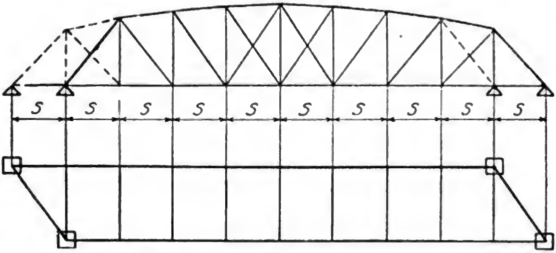


FIG. 22*nn*. Arrangement of Trusses for a Skew Span with Polygonal Top Chords.

In skew spans having polygonal top chords, it is necessary for the members of the two trusses to be parallel in each panel. The panel points between the hips at the acute corners should, therefore, lie on the same curve. The arrangement should be as shown in Fig. 22*nn*. In case the skew extends over more than one panel, the top chord at the acute ends of the span should be made straight for this length on account of the complications entailed in the details of the upper lateral system and in those of the vertical sway bracing.

When a parabolic curve is used for a span with polygonal top chords, the lengths of the various posts can be computed as follows: For a truss with an odd number of panels of equal length, the differences between the lengths of successive posts, beginning at the centre of the span, are proportional to the numbers 1, 2, 3, 4, 5, etc.; for a truss with an even number of panels of equal length and all top-chord members inclined, the said differences are proportional to the numbers 1, 3, 5, 7, 9, etc.; and for a truss with an even number of panels of equal length and all top-chord members except the centre two inclined, they are proportional to the numbers 0, 3, 5, 7, 9, etc. To illustrate the application of these series, let us find the lengths of the posts in an eleven-panel through truss in which the hip depth is thirty-four (34) feet and the centre depth forty-four (44) feet, the top chord being nine panels long. There are evidently four inclined chord members in each half of the truss. The differences between the lengths of successive posts, being proportional to the numbers 1, 2, 3, and 4, may be represented by x, $2x$, $3x$, and $4x$. The difference between the centre depth and hip depth, or ten (10) feet, is then equal to $x + 2x + 3x + 4x$, or $10x$, so that the value of x is one foot. The differences between the lengths of successive posts are thus found to be one (1) foot, two (2) feet, three (3) feet, and four (4) feet, so that the lengths of the various posts are forty-four (44) feet, forty-three (43) feet, forty-one (41) feet, thirty-eight (38) feet, and thirty-four (34) feet.

The question of the riveted versus the pin-connected truss is fully discussed in Chapter XXXII. As explained there, each type has its own place in good construction and should be employed where its advantages are preponderant and well defined.

In Chapter LIII are discussed various items of economy which affect the truss in itself as well as in its relation to the structure as a whole; and in Chapter LII is given a dissertation on the principles governing the æsthetic treatment of bridges. Arrangements of the roadways to accommodate the various classes of traffic crossing the structure are fully outlined in Chapter XVIII; while Chapter LIV gives a list of the factors affecting the layouts of bridges and a complete discussion regarding them. This last chapter is especially important, as upon the layout adopted often depends the success or failure of a bridge enterprise. It covers the most important part in the preparation of the design. The various loads—dead, live, impact, centrifugal, wind, vibration, and traction—are treated in Chapters V to IX, inclusive, while the actual loads to be used in designing are specified in Chapter LXXVIII.

Wind loads rarely affect any of the truss members in ordinary spans except the inclined end posts in through bridges. In long spans, however, it is necessary to consider the effect of wind. A test of the bottom chord section $L_1 L_2$ in the Pratt truss or $L_3 L_4$ in the Petit truss will indicate whether the wind stresses influence the sections. The weights for the floor system, trusses, and bracing to be used in the preparation of preliminary estimates and final designs for various kinds of structures are given on the diagrams in Chapter LV. Stress computations are considered to a certain extent in Chapter X, but the reader is referred to such standard texts as "Roofs and Bridges," by Merriman and Jacoby, and "Modern Framed Structures," by Johnson, Bryan, and Turneaure, for a complete treatment of the subject. Combinations of stresses and secondary stresses are discussed in Chapters XIII and XI, respectively. It is to be remembered, however, that secondary stresses should be avoided and minimized just as far as possible, rather than figured and cared for by the provision of extra metal. The actual designing and detailing will be governed by the specifications in Chapter LXXVIII. However, before preparing a design or the detailed plans, an engineer should be fully conversant with the cardinal principles laid down in Chapter XV and he should likewise be familiar with such points as affect the shopwork, which points are clearly stated in Chapter XVII. The dimensioning of truss members for camber is treated in Chapter XXXIII.

The design and details of the floor and floor system are taken care of in Chapter XIX, while that of the laterals is given in Chapter XX. The trusses alone will be considered in this chapter. In order to handle the subject to the best advantage, riveted and pin-connected trusses will be treated separately. The riveted truss will be discussed in full, and the details in which the pin-connected truss differs from it will be taken up afterward.

The truss consists of two principal parts—the chords, which take the stresses due to bending, and the web, which carries the shear. This division is not strictly correct, as polygonal chords also take shear; however, it is sufficiently definite for all practical purposes, and will be so assumed in this treatise. The chords consist of the members forming the upper and lower outlines of the truss; while the web comprises the diagonals (both tension and compression), the posts, the hangers, and the inclined or vertical end posts. The inclined end posts are really a continuation of the top chords, and generally have sections of the same outline and construction.

The make-up of the members forming the truss depends largely upon their function as well as upon the stresses to which they are subjected. Figs. 22*oo* to 22*aaa*, inclusive, show the arrangement of various sections. The full lines of these figures represent the fundamental elements which are always present; while the dotted lines indicate portions which may be added to give heavier sections, or for some other special reason.

The smallest section used consists of two angles back to back (Fig. 22*oo*) riveted together about every twelve (12) inches with intervening washers of a thickness equal to that of the gusset-plates. This section is employed only in the lightest highway spans of the pony-truss type, and sometimes in the open-webbed girders of elevated railways. The web members of such trusses are composed of unequal legged angles with the shorter legs outstanding in order to obtain radii of gyration approximately equal in the two directions. This is necessary for the compression diagonals, although it is not as essential in the tension members.

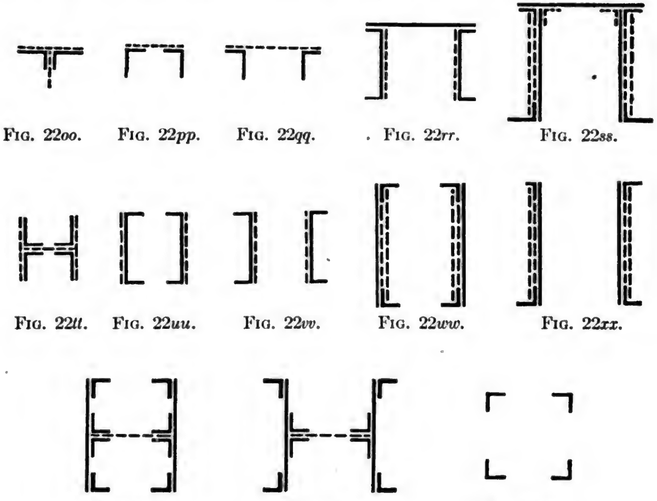


FIG. 22*oo*. FIG. 22*pp*. FIG. 22*qq*. FIG. 22*rr*. FIG. 22*ss*.

FIG. 22*tt*. FIG. 22*uu*. FIG. 22*vv*. FIG. 22*ww*. FIG. 22*xx*.

FIG. 22*yy*. FIG. 22*zz*. FIG. 22*aaa*.

Sections for Truss Members.

There is, however, an advantage gained by placing the angles in this manner even in tension members; viz., that because the adjacent angle legs are riveted directly to the gusset-plates, the wider these legs the more compact the connection. Lug angles should not be used to develop the outstanding legs of the angles; instead, the sections should be increased so that they will not be overstressed on account of the eccentricity of the rivet groups. The chords for deck structures are usually arranged in the same manner as the web members. However, in pony trusses it is necessary to stiffen the top chords transversely as much as possible, and in such members the longer legs should be turned out. A cover-plate can sometimes be added advantageously. The chords and the inclined end posts are frequently built of two angles back to back with an intervening plate which projects sufficiently beyond the tip of the adjacent legs to permit the connection to it of the web members directly. This avoids the use of gusset-plates; but, of course, it can be employed only for trusses with light web stresses.

The thickness of the gusset-plates in these light trusses should be gauged by the details at the joint, as it is often more economical, as well as more satisfactory, to use thick plates and a correspondingly small number of rivets. The thickness, however, need not exceed that of both the angles combined, neither need it be such as to give a bearing value for rivets greater than the double shear. As the specifications require the angles to be not less than three-quarters (¾) of an inch from back to back, it is very easy to arrange this detail as desired.

The gravity lines of all members coming together at an apex should meet in a point. This is an important fact which should never be forgotten or ignored.

In making up the members of a truss of the type just discussed, as few different sections as are consistent with reasonable economy should be selected. It should be remembered that the use of a large variety of sections in a small structure will often prevent quick deliveries of metal by the mills, and in addition it will increase the amount and cost of the shopwork. In chord members it is sometimes more economical to use the same section throughout several panels and thus avoid the splices required when different sections are employed.

In order to gain greater rigidity in these short, light spans, the angles forming the members are frequently spread, those taking compression being laced and those under tension being connected with batten plates about every three (3) feet. In the bottom chords and the web members the angles are generally turned in as shown in Fig. 22*pp*; while in the top chords and the inclined end posts, which usually have a cover-plate, they are turned out, as shown in Fig. 22*qq*, in order to increase the stiffness of these members. The various members are connected by two gusset-plates at each joint.

Except for the small pony-truss and the open-webbed girder spans just discussed, the truss members are nearly always made up of more substantial sections than those previously described. As a rule, these are balanced, and consist of two leaves, or parts, tied together with batten plates, lacing, cover-plates, and in some cases solid diaphragms. For light highway through and deck spans, some of the members are frequently built of two angles connected by lacing or batten plates, as described in the previous paragraph, for the reason that the stresses are too small to warrant anything else; but in the more important highway structures, and in all railway truss bridges, the heavier balanced sections are used. These consist of two pairs of angles in the form of an I and connected with lacing, batten plates, or a solid web (Fig. 22*tt*); two rolled or built channels in the form of a box with the flanges turned either in or out and connected with lacing or batten plates (Figs. 22*uu* to 22*xx*, inclusive) or with a laced or solid diaphragm down the centre (Figs. 22*yy* and 22*zz*); or two rolled or built channels with a cover-plate connecting the top flanges and with lacing on the bottom (Figs. 22*rr* and 22*ss*). In the determination of the make-up and outline of such members, various factors must be taken into account, especially the functions of the members. Members acting in compression must be stiff as a whole as well as in the individual parts that form the section. The metal should be concentrated near the periphery so as to give large radii of gyration; and it should be distributed so that the moment of inertia will be about the same in the two directions, unless the unbraced lengths are different. The metal must not be spread out to such an extent that the component parts are weak in themselves even though thoroughly braced. In no case must any individual part of a member be weaker than the member as a whole; and in fact it should be somewhat stronger, as a secondary effect takes place in each part that acts in itself as a small column. Tension members do not, as a rule, need special consideration, except to provide for good details and to ensure that the limiting value of the l/r ratio for tension members is not exceeded. They are always made to conform with the rest of the truss. When they take compression as well as tension, they must be considered from this standpoint as well. Chords acting as beams must be designed for bending as well as for direct stresses; and it is generally necessary to make them deeper than they would be otherwise.

The sections should be arranged so as to provide the simplest and most satisfactory details in the span as a whole, and at the same time so as to make the shopwork as easy as possible. These two conditions cannot always be fulfilled; and when such is the case, the proper detailing of the span should receive the first consideration. One of the chief points of contention between the shops and the bridge engineer is in regard to the turning in or out of the angles in built-up box sections. On account of the difficulty the shops have in riveting the lacing when the angles are turned in, the latter should be turned out if it is convenient to do so. Care should be taken, however, in so doing, to see that the members do not become excessively wide and consequently require very heavy lacing. Questions of erection must also be considered in this connection.

In through structures the angles of the top chords and end posts are almost invariably turned out, and those of the other web members and of the bottom chords turned in. Especially is this true where the ordinary floor system of stringers and floor-beams is employed, as this construction provides a smooth surface for the end connections of the latter. If the angles of the bottom chord are turned outward, the floor-beams have to be slotted or cut out at the ends to receive them. If the angles of the posts are turned out, they will interfere with the floor-beams, if these should have to be erected after the trusses are up. Also the end details of the web members are not satisfactory when the angles are turned out, as this necessitates running the leaves which make up the sections outside of the gussets, and thus prevents placing the tie plates close to the ends. This is especially important in compression members, where it is necessary for the two halves to be thoroughly connected throughout their full length.

Under certain conditions the angles of the bottom chords of through bridges can be turned out without the foregoing disadvantages. This was done by Boller, Hodge, and Baird in the Congress Street Bridge at Troy, N. Y., described in the *Engineering News* of March 25, 1915. In this case the floor panels had a constant length of ten (10) feet throughout, while the truss panels varied from the end to the centre of span. Rolled I-beam stringers and shallow built floor-beams were adopted; and the latter were riveted to the bottom chords directly. The chords and the floor-beams were of the same depth, and the corners of the latter were coped for the outstanding angles of the former, the end connections being made in the depth of the web between the angles. The web members had their flanges turned in. The same construction can be employed on through bridges with shallow floors where either troughs or I-beams are riveted between the bottom chords or where the floor system is connected below them.

In deck structures the bottom chord angles can be turned out, with the additional advantage of providing a good connection for the bottom laterals. The flanges of the web members are sometimes turned out; but for the reasons mentioned previously they should be turned in. When the floor lies entirely between the trusses, the floor-beams are dropped just below the top chord, and the angles of the latter are turned out. The same arrangement can be adopted when additional roadways are placed entirely outside of the trusses. When, however, the roadway extends over the latter, it may be necessary to bring the cross girders to the top of the chords, in which case the angles should be turned in. In case a shallow floor construction is employed, as described for through bridges, the chord angles can be turned out; and this same arrangement can be adopted for railway spans in which the ties rest directly on the top chords.

When I-sections are used for the bottom chords and web members, the widths of these may be dependent upon their own make-up or upon that of the top chord. When box sections are employed, the width is generally determined by the members which have their flanges turned in. The distance between the tips of such flanges should never be less than five (5) inches, as this space is necessary for riveting on the lacing and stay plates and for painting. In fact, it should preferably be increased to five and one-half (5½) or even six (6) inches. The widths of compression members should be consistent with their lengths. For top chords the ratio of the length to the least radius of gyration usually ranges from thirty (30) to forty (40), although for short spans it may run as high as sixty (60) or even seventy (70). For web members this ratio usually ranges from fifty (50) to ninety (90), at times reaching as high as one hundred (100). The maximum allowable limits are given in the specifications of Chapter LXXVIII.

The depths of the main members depend to a large extent upon the stresses they have to carry. The metal must not be piled up unduly, but should be so disposed that a well-proportioned section will be the result. In addition due consideration must be given to the truss as a whole. It should have a substantial appearance, which can be attained only by a proper proportioning of the members. With built channel sections the top and the bottom chords are generally made of about the same depth, the bottom chord being the deeper by a few inches, as there is more metal in its webs than in those of the top chord, due to the use in the latter of the cover-plate and heavy bottom angles and to the necessity for considering both the ratio of l over r and the general restrictions governing the make-up of compression members. When rolled channels are employed, the same depths are almost invariably adopted for both chords. This is likewise true where cover-plates are not used. When either chord is subjected to bending in addition to the direct stresses, it must be treated as a beam; and this requires a considerably greater depth than would be necessary otherwise. The inclined end posts generally have the same depth as the top chords. The end web members, likewise, have about the same depth as the top or bottom chord, although sometimes they are a trifle shallower. The depths of the intermediate web members diminish toward the middle of the span on account of the decrease in the sectional areas. Hangers, sub-members, and stiffening struts are made of such depths as are necessary for their particular requirements. Where floor-beams attach to these they must be wide enough to provide for a good connection for the beam itself as well as for that of the member to the gusset-plate. If sufficient rivets for both cases cannot be secured in the width of the floor-beam connection angles, the member must be made wide enough to allow for whatever rivets are needed.

The most economical of the built sections within reasonable limits is the four angle I, shown in Fig. 22*tt*, with a single line of lacing, batten plates, or a web-plate. Side-plates are added, as indicated, for the heavier sections. It has the fewest parts, is compact and rigid, is open so that both the shopwork and the fieldwork are the simplest possible to obtain, and is easily painted and maintained. It is well suited for the tension members of trusses of medium span-length, as well as for struts and hangers employed to support the main members. It is likewise adapted to short posts, although the section is not the best for compression members when the length is considerable. When used for the bottom chord, the web plate should not be employed, the two halves being connected by batten plates or lacing; as otherwise a trough is formed which will collect dirt and water and thus involve trouble in protecting the steelwork. In hangers or posts carrying floor-beams directly, the employment of a web should be carefully considered, as a solid plate is needed in any case for a distance equal to the depth of the floor-beam, and it might be more economical to extend it over the full length of the member and make it part of the section. This is especially true in very narrow members, in which it is not infrequent to find the web-plate weighing less than lacing, which, of course, cannot be counted in the section. The same also holds true for other tension and compression members.

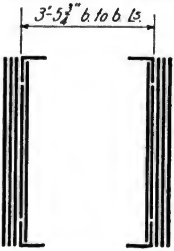
When heavier sections than the four-angle I are required for the bottom chord and the web members, those shown in Figs. 22*uu* to 22*xx* are employed. These are either laced or connected with batten plates on the open faces. For small sections, rolled channels with flanges turned either in or out are used; and when a greater area is needed than the channels alone will provide, side-plates are added. This permits of varying the section without changing the depth, and is of particular advantage in the bottom chord. However, the use of such side-plates piles up the metal considerably, which results in a material loss of net section in tension members, and a reduction in the radius of gyration for compression members; so that ordinarily, where the maximum rolled sections do not give sufficient area without additional plates, the built channel should be resorted to.

The built channels shown in Figs. 22*ww* and 22*xx* lend themselves to any variations whatsoever, the only limitations being the sizes and lengths of the rolled sections. Various factors, however, will determine the best arrangement for any given truss.

In the first place, the sections must be in conformity with the rest of the structure. Short light spans will have small sections, while the longer and heavier trusses will have larger ones. The former, as a rule, involve no trouble in the determination of their make-up, while the latter may give considerable. As most of the metal is in the side-plates, the possible variations for any given area are mainly in the thickness and depth of these. The thickness is limited only by the permissible grip for rivets—the larger the rivet, the greater being the limit for grip. These limits are fixed by the specifications. The depth, on the other hand, should be kept down to prevent excessive secondary stresses. The length of the panel adopted for the truss plays an important part in this; for the longer the panel, the greater may the depth be made consistently. In the 423′ spans of the Fratt Bridge over the Missouri River at Kansas City, Mo., the section shown in Fig. 22*bbb* was used for one of the bottom chords, while that illustrated in Fig. 22*ccc* indicates the make-up of one of the bottom chord sections for the 287′ span of the O.-W. R.R. & N. Co.'s Bridge over the Willamette River at Portland, Ore. These figures represent typical sections for heavy built-up members.

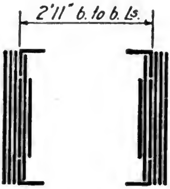
As few as possible of the different sizes of rolled sections, as well as of sections of members themselves, should be adopted in proportioning trusses. Moreover, special sections should be avoided, as the mills will not make quick deliveries on them unless the tonnage is large; and even in such a case they may object to doing so. In this connection it might be well to state that the list of standard plates given in Chapter XVII should be kept in mind.

Full-depth web-plates in truss members should be employed, unless it proves uneconomical to do so; because plates between the angles increase the shopwork in more than one respect. These plates require extra rivets, as it is necessary to tack them to the main webs along each edge, and closer spacing than that ordinarily employed for stitch rivets will have to be resorted to. Moreover, these rivets will reduce the net sections of tension members to a greater extent than is the case where only full-depth webs are employed. In compression members such narrow plates tend to reduce the radius of gyration on account of the concentration of metal near the centre of gravity. It should not be lost sight of, though, that these plates provide an excellent means of varying the section in continuous members, as is explained later. When employed, their width, ordinarily, should be three quarters (¾) of an inch less than the clear distance between the angles in order to provide for the overrun in the latter as well as for the weave in the plates. In very heavy sections, it may be advisable to increase this to as much as one and one-quarter (1¼) inches. These two figures are suggested in order to permit the ordering of these narrower plates in widths of inches or half-inches; for the depths of the web-plates are always made in even



6 Pls. 60″ × 15/16″
2 Pls. 44″ × ¾″
2 Pls. 58″ × ⅝″
4 Ls 8″ × 8″ × ¾″

FIG. 22*bbb*. Typical Bottom Chord Section for the 423-foot Fixed Spans of the Fratt Bridge over the Missouri River at Kansas City, Mo.



6 Pls. 42″ × ⅝″
2 Pls. 26″ × ¾″
2 Pls. 39½″ × 11/16″
2 Pls. 26″ × ⅝″
4 Ls 8″ × 8″ × ¾″

FIG. 22*ccc*. Typical Bottom Chord Section for the 287-foot Span of the O.-W. R. R. & N. Co.'s Bridge over the Willamette River at Portland, Ore.

inches, and the distances back to back of angles should be one-quarter (¼) inch greater than the said depths of the web-plates.

As few web-plates as possible are advocated by some prominent engineers even to the extent of using the thickest obtainable and drilling them in preference to employing a larger number of thinner plates and sub-punching and reaming them. There are certain definite advantages in adopting the former practice, while there are likewise decided disadvantages. By having a single web-plate for each leaf, all stitch rivets except those connecting the angles to the plates are avoided. This saves material in tension members in that fewer holes are taken out of the section, while it also economizes in the shopwork in that it reduces the number of parts to be handled as well as the number of rivets to be driven. The first-mentioned economy may not be realized on account of the rivets required in the first row in the splice-plates or the gusset-plates, as the case may be; for, as mentioned later, this point generally determines the net section of the tension members. Its principal disadvantage is that it requires full butt splices at every point where the section changes in continuous members; and this augments the amount of material necessary for the truss. In addition, in the heavier sections it causes an increased thickness of the metal at splices, which may be too great for the rivet grips specified. The most economical splice is the shingle splice, which can be employed only when the webs are built up of a number of plates; and this splice has the advantage, in heavy trusses, of keeping down the thickness of the metal.

In the web members of trusses with single gusset-plates on each side at the panel-points, the single webs will undoubtedly be economical; and the same will be true in other cases of moderate sections where the advantages of shingle splicing are not so pronounced. However, in heavy trusses where it is necessary to use several gusset-plates per side and where the sections change considerably, the webs should be built of a convenient number of plates. Naturally, the heavier the sections the thicker can the webs be made economically, just as under such conditions the size of the rivets is increased.

It is this same condition, the necessity for the use of heavy members, that merits the consideration of inside plates between the angles. This is of particular importance in the bottom chords of long spans where a large variation takes place between the centre and the end sections, and where considerable difficulty is encountered at times in arranging these sections to the best advantage. Where, for other reasons, stitch rivets are unavoidable, there seems to be no reason for objecting to the plates between the angles, except that a few extra rivets are required. Even this might not be necessary, if web-plates are added over the angle legs.

The fact that the procurable lengths of plates decrease with the increase in size must be given careful consideration. With the shorter plates the splices might not work out satisfactorily, and in addition there might be a loss in economy on account of a possible increase in their number. Another point that should not be lost sight of is the fact that it is sometimes more economical to use the same section over more than one panel length than to introduce an extra splice. This, however, should be verified for any particular case.

The angles for built-up box sections depend on the size of the latter. For the smaller ones 3½″ × 3½″ angles are largely used; but as the sections increase, 4″ × 4″ and even larger angles are employed. The smaller the angles the smaller will need to be the width of the member in order to accommodate the fabrication; and this is of importance in short spans, except where an increased width would not prove uneconomical. The use of 4″ × 3″ or 6″ × 3½″ angles with the shorter legs outstanding is frequently advisable on this account. Large angles, on the other hand, are advantageous for compression members, as their use increases the radius of gyration with a consequent reduction in the area required. Another point that must be considered is the employment of web- or splice-plates over the legs of the angles. Where this occurs there must be sufficient space for the rivets through the horizontal legs of the angles; and the said legs must be long enough to provide therefor. In the heaviest sections the adoption of four angles at top and four at bottom has sometimes been resorted to; but the author does not altogether like this method on account of the clumsy details at the joints necessitated by such an arrangement. In members carrying transverse loads it must be remembered that the angles act as do the flanges in ordinary girders; and for this reason heavy angles are advisable.

The sections shown in Figs. 22*yy* and 22*zz*, consisting of built channels with a centre horizontal diaphragm of angles and lacing or of angles and a web-plate, are sometimes used for compression members. When employed, the open faces are connected by batten plates. These form very rigid sections, and are especially adapted for large and important members.

For struts of secondary importance where neither the four-angle I nor the rolled channel section previously described is stiff enough, the four-angle section shown in Fig. 22*aaa* is well suited. This is laced on all four faces; and it lends itself very readily to any desired outline.

The sections shown in Figs. 22*rr* and 22*ss* are used exclusively for the top chords and the end posts—that shown in Fig. 22*rr* being employed for light spans, and that in Fig. 22*ss* for the heavier trusses. Their make-up is similar to that of the channel sections previously described; and the same statements hold in respect to both. The cover-plate is added to give rigidity to the chord, and especially to the end post, which, in through spans, is subjected to transverse bending as well as to direct stresses.

The width of the cover-plate is determined by that of the web members in the smaller trusses, while in the larger ones it depends on the proportions of the section itself. Ordinarily it is made from one (1) to eight (8) inches more than the depth of the member. The greatest difference is necessary in the small sections where the webs are composed of rolled channels; and as the sections increase in area this difference becomes smaller, until in the largest members the width of the cover-plate is made about equal to the depth of the side-plates.

The cover-plate should always extend beyond the edges of the top angles enough to take care of any overrun in the latter. For small sections its width should exceed by at least one-half (½) inch the distance from tip to tip of angles; and for the large ones, this allowance should be increased to one (1) inch.

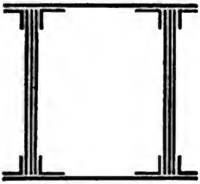
Except in such abnormal constructions as pony trusses, in which excessive chord width is essential, the radius of gyration of the top-chord section about its vertical axis should be only a little greater than that about the horizontal axis. It is advantageous to have it somewhat larger on account of the bending in the inclined end posts.

It is desirable to keep down the thickness of the cover-plate in order to avoid the concentration of a large amount of metal at the top, and the consequent difficulty of balancing with a corresponding amount at the bottom. The specifications, however, prevent using a cover-plate having a thickness less than one-fortieth ($\frac{1}{40}$) of the distance between the inner rivet lines. To overcome this unbalanced condition, the bottom angles are made heavy with wide outstanding legs. This lowers the centre of gravity of the section, which should be maintained as near the centre of figure as possible. Angles, 5″ × 3½″, 6″ × 4″, and 6″ × 6″, are ordinarily used for these flanges, with 8″ × 8″ angles for the heaviest sections. Generally only two top angles are employed, and these are 3½″ × 3½″ or 4″ ×4″ in size, but for heavy sections heavier angles are necessary. Moreover, the addition of inside angles, of the same size as the outside, is sometimes resorted to, especially when the chord is rather wide and when the use of the angles will reduce the thickness of the cover-plate. The author does not follow this practice except in the heavier sections on account of the trouble it presents in the details at the splices and joints. Inside bottom angles, likewise, have been used; but, as in the case of the top angles, the author resorts to them only in the heaviest sections, and even then he prefers to omit them.

Fig. 22*ddd* shows the section of the special type of top chord evolved by the author early in 1907 for the 423′ fixed spans of the Fratt Bridge over the Missouri River at Kansas City, Mo.; but it was not used in the final design because it was considered by the shops to be too great an innovation. It consists of a closed-box section made up of side-plates, top and bottom pairs of angles on each leaf, and top and bottom cover-plates. The section was stiffened with cross diaphragms about every ten (10) feet. It was large enough to permit a man to pass through easily for both inspection and painting; and openings were left in the cross diaphragms to give passageway over the full length of the span. Closed sections of a somewhat similar make-up have lately been used in the designs for the Quebec Cantilever and the Hell Gate Arch bridges.

In determining the sections of the truss members, after the stresses have been figured, it is necessary to consider all of the members together to a greater or less extent. The sizes of these and the question of turning the flange angles in or out are important factors in the problem; but the splicing and the gusset connections are even more important. Special consideration must also be given to trusses with polygonal top chords, where the splices have to be made at the joints.

In small trusses where rolled channels and four-angle I's are used, single gusset-plates, one on each side of the joint, are all that are necessary; and these usually are of minimum sections. The backs of the channels in the top and the bottom chords are lined up, and the gusset-plates are placed against them. The in-to-in of the gussets is kept constant, as is also the out-to-out or in-to-in of all web members, depending on whether the channels are turned in or out. The same arrangement is employed where built sections consisting of single plates and a pair of angles are used.



2 Cov. Pls. 61″ × $\frac{11}{16}$″
4 Web Pls. 54″ × $\frac{11}{16}$″
2 Web Pls. 54″ × $\frac{5}{8}$″
4 L• 6 × 6″ × $\frac{3}{4}$″
4 L• 8 × 6″ × $\frac{1}{2}$″

FIG. 22*ddd*. Proposed Top Chord Section for the 423-foot Spans of the Fratt Bridge over the Missouri River at Kansas City, Mo.

However, where compound webs are advisable, a different situation arises. In this case it is necessary to arrange the plates and angles for simple and economical splicing and for effective gusset connections. While single gussets are sufficient for ordinary spans except at the hip and the end bottom chord joints, their thickness generally varies, decreasing from the end of the span to the centre. As it is desirable to keep the web members the same distance from out to out where the angles are turned in, the distance in to in of gussets should be made constant. This is very simple where the flanges are turned in, as the chords can then be arranged in the same manner as are the web members by fixing the distance from out to out of web-plates. In this case the make-up of the truss members is affected only by the splices.

The minimum and the maximum sections of the chords should be determined first, and those intervening should then be arranged to correspond. Naturally, there has to be a cutting-and-trying until the best layout is secured. The end or smallest member will generally consist of two plates and four angles. If practicable, the same section of plates and angles should be used throughout the entire chord, the increase in areas being made by the addition of full-depth plates and plates between the angles. The latter should, if possible, be of the same thickness as the angles so as to avoid the use of thin fillers at the splices. The additional full-depth plates should, preferably, likewise be of the same thickness as the angles. As full-depth plates are added, the angles are offset toward the centre; and, consequently, the middle section will determine the width of the chords as well as that of the web members. This method of building the sections of the chords permits the shingling of the plates and angles at the splices, which reduces the thickness of metal at these points as well as the amount thereof required for splicing. Sometimes the minimum chord section is composed of four webs and four angles. In this case the method just outlined should be followed.

Where the angles of the chords are turned out, a somewhat similar procedure will be advisable. In this case, though, the angles should, preferably, be kept in line, and the plates offset. This arrangement is of greater value in the top chords and end posts, which have cover-plates, than in the bottom chords where no cover-plates are used, both because of appearance and because of the advantage of keeping the gauges in the angles and top plate constant throughout. As the chords increase and the gussets decrease in thickness from the end to the centre, a balance may be struck for maintaining a uniform distance from the back of the angles to the inside of the gussets without making the latter heavier than necessary. If this cannot be done, certain of the gussets can be slightly increased in thickness, or thin fillers can be inserted between them and the chords. Fillers, however, are a nuisance both in the shops and in the field; and they should be avoided just as far as possible. On the other hand, though, metal should not be wasted in the gussets to any extent in order to avoid fillers.

As far as practicable, the make-up of the chords should be such that there will be the minimum amount of splicing of parts necessary in the shops. Usually the field splices are placed in alternate panels, and sometimes in every third panel. This introduces one or more changes in the section between these points. Where it is economical to do so, the same section can be extended over more than one panel and the shop splice avoided.

In heavy trusses two or more gusset-plates per side are needed in order to transfer the stresses to the various members. Where the chord plates can be secured in lengths sufficient to extend over more than one panel, they are spliced outside of the joints. The increases in section can be spliced into the gussets very economically. The web members should be shingled-in with the gussets in the same manner as is done in the chord splices. When the chord plates are so large that they cannot be secured in lengths greater than that of the ordinary panel, it is necessary to splice them at all joints; and in this case the web-plates of all the members meeting at a point should be arranged so that they can be shingled-in with the gusset-plates, the latter being used for splice material. The former arrangement was employed on the O.-W. R.R. & N. Co.'s bridge over the Willamette River at Portland, Ore., while the latter was adopted for the unusually heavy trusses of the 423′ spans of the before-mentioned Missouri River Bridge at Kansas City, Mo. In such heavy structures it is almost impossible for the designer to arrange the sections unless he makes a particular study of the joints and the gusset-plates required. As a rule, it is better for him to determine the outline of the sections and their tentative make-up, leaving the final arrangement to the detailer.

For trusses with polygonal top chords, it is always necessary to splice these at the panel-points. Full butt-splices are preferable where the specifications permit relying upon the abutting of a given percentage of the section. Where a full splice is required, the gusset-plates should be used for splicing the adjoining sections in the same manner as was just described. The web and the bottom chord members do not present any different problems from those met with in the trusses with parallel chords.

Fig. 22*eee* shows the details of the trusses for a 271′ span which was designed by the author's firm. This indicates the manner of building the truss sections as just described. Fig. 22*fff* illustrates the details of the floor system, laterals, and shoes of the same span.

In proportioning the members of any truss, sections compiled from previous structures should be consulted; and if such a compilation is not available, some standard work such as Osborn's Tables or Merriman and Jacoby's "Roofs and Bridges" will prove very useful. Kunz in his recent book, "Design of Steel Bridges," gives several excellent tables of the ordinary sections of truss members, covering quite a wide range of areas. The make-up and the area of each are shown, as well as the radius of gyration for the compression members. Where such tables are not at hand, a competent designer will have no difficulty in determining the sections. The compression members require the greatest amount of work, as it is necessary to know the radius of gyration, at least approximately, before the unit stress can be determined and the area computed; and this cannot be settled until the section is known. Table 22*a*, giving the approximate radii of gyration about the two principal axes of all forms of sections ordinarily used for compression members, will prove very helpful. From Fig. 16*d* the location of the point of contraflexure in the end posts of through bridges can be taken. Tension members of any fixed net section should have the minimum gross area practicable. This

will be determined by the first row of rivets in the splice or the gusset-plates. Fig. 16*e* enables the net section for various spacings of rivet lines and gauges to be readily computed.

In this connection it might be well to call attention to the folly of filling the members with unnecessary rivets. Such procedure simply increases the shop cost without any advantage to the structure. The specifications should not be ignored to the extent of using too few rivets; but neither should an excessive number be employed. This is a common fault of a great many draftsmen, especially beginners. Moreover, the rivet spacing should be arranged for multiple punching. No member should be extended into another unless the two are at right angles to each other; consequently the diagonals should not be run inside of the chords. Ordinarily, there is no particular advantage in extending the posts thereinto; and in all probability there will really be a waste of material in doing so, as the connection can be made satisfactorily below the chords, thus fitting in better with that for the diagonal members. However, in deck structures with the floor system attached below the chord, the post should be extended into the latter in order to support it thoroughly. In Triangular trusses the vertical members sustaining the chords should be extended thereinto whenever this is possible, the gusset-plates being omitted.

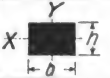
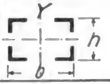
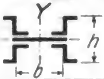
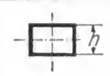
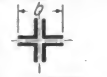
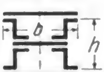
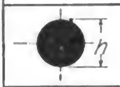
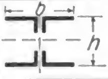
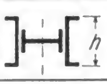
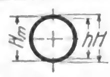
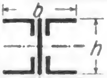
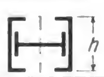
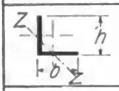
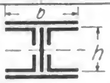
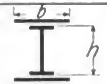
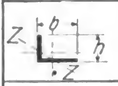
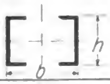
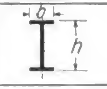
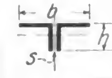
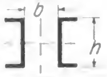
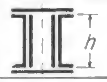
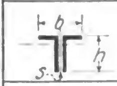
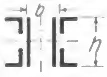
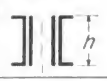
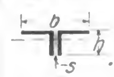
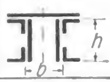
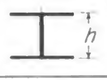
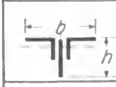
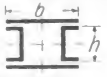
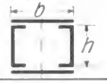
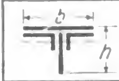
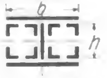
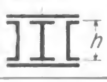
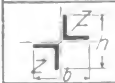
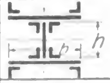
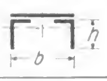
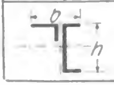
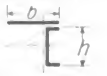
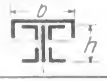
Another point that needs consideration in the designing of the chords is the connection of the laterals. Where their intersection does not fall on the centre line of the chord, the effect of the eccentric wind stresses may increase the section slightly, although this rarely is the case except in very long spans. Moreover, only the loaded chord, as a rule, needs attention. A short investigation will tell whether such is the case or not. Chapter XX on "Laterals and Sway Bracing" discusses this point fully.

The details of the truss members are as important as the sections themselves, if not more so; for experience has shown that most bridge failures have been due to faulty details rather than to incorrect proportioning of main members, although the latter has not been lacking altogether. Some of the most important recent considerations have been along the lines of the bracing necessary to make the component parts of a member act as a whole; and especially is this true in regard to compression members.

The parts composing the different truss members are variously connected, depending on the function of the latter as well as on the size of the sections. The two leaves of compression members are connected by lacing at both top and bottom, by a central solid diaphragm with batten-plates at top and bottom, by a top cover-plate with bottom lacing, or by cover-plates at top and bottom. Lacing on the two open faces is used for compression web members, and sometimes for the top chords, when these are formed of two rolled or built channels of ordinary section.

TABLE 22*a*.

APPROXIMATE RADII OF GYRATION

 $r_x = 0.29h$ $r_y = 0.29b$	 $r_x = 0.42h$ $r_y = 0.42b$	 $r_x = 0.31h$ $r_y = 0.48b$
 $r_x = 0.40h$ $h = \text{mean } h$	 r_y = same as for 2 ∠s	 $r_x = 0.37h$ $r_y = 0.28b$
 $r_x = 0.25h$	 $r_x = 0.42h$ r_y = same as for 2 ∠s	 $r_x = 0.31h$
 $r = \sqrt{\frac{H^2+h^2}{16}}$ $r = 0.35\,H_m$	 $r_x = 0.39h$ $r_y = 0.21b$	 $r_x = 0.31h$
 $r_x = 0.31h$ $r_y = 0.31b$ $r_z = 0.197h$	 $r_x = 0.45h$ $r_y = 0.235b$	 $r_x = 0.40h$ $r_y = 0.21b$
 $r_x = 0.29h$ $r_y = 0.32b$ $r_z = 0.18\,\frac{h+b}{2}$	 $r_x = 0.36h$ $r_y = 0.45b$	 $r_x = 0.38h$ $r_y = 0.22b$
 $r_x = 0.31h$ $r_y = 0.215b$ $= b(0.21 + 0.02s)$	 $r_x = 0.36h$ $r_y\ 0.60b$	 $r_x = 0.39h$
 $r_x = 0.32h$ $r_y = 0.21b$ $= b(0.19 + 0.02s)$	 $r_x = 0.36h$ $r_y = 0.53b$	 $r_x = 0.35h$
 $r_x = 0.29h$ $r_y = 0.24b$ $= b(0.23 + 0.02s)$	 $r_x = 0.39h$ $r_y = 0.55b$	 $r_x = 0.435h$ $r_y = 0.25b$
 $r_x = 0.30h$ $r_y = 0.17b$	 $r_x = 0.42h$ $r_y = 0.32b$	 $r_x = 0.42h$
 $r_x = 0.25h$ $r_y = 0.21b$	 $r_x = 0.44h$ $r_y = 0.28b$	 $r_x = 0.42h$
 $r_x = 0.21h$ $r_y = 0.21b$ $r_z = 0.19h$	 $r_x = 0.50h$ $r_y = 0.28b$	 $r_x = 0.285h$ $r_y = 0.37b$
 $r_x = 0.38h$ $r_y = 0.19b$	 $r_x = 0.39h$ $r_y = 0.21b$	 $r_x = 0.42h$ $r_y\ 0.23b$

It is figured according to the formula given in the specifications of Chapter LXXVIII, which specifications fix also the minimum sections allowable. In Chapter XVI on "Detailing in General" the subject of lacing is very fully discussed. In Figs. 16*a* and 16*b* are given curves for the weights of various types, both single and double, for different distances from centre to centre of rivet lines; and Fig. 16*c* can be employed for the design of lacing. From these diagrams the most economical lacing for any given section can readily be determined. The top chords generally have cover-plates, with lacing in the bottom plane. In this case the lacing is designed in the same way as it is when used on both faces, the cover-plate being considered as one of the planes of lacing. For the heavier built sections many engineers advocate the central solid diaphragm consisting of four angles and a web with batten-plates on the open faces. Sometimes lacing replaces the web-plate. Either detail makes an exceptionally good section. In the same way the use of cover-plates at top and bottom have come to replace the lacing in the very heavy sections, as they can be depended upon to a much greater extent than the lacing, especially when properly stiffened; and they can also be counted in as part of the area of the member. Lacing, when employed, should always be arranged so as to permit of easy access for painting. For this purpose a cylinder four (4) inches in diameter should be capable of insertion between any set of bars and the main section.

The two leaves of tension web members are usually connected by batten-plates except where counter-stresses compel them to take compression as well as tension, in which case they are laced like ordinary compression members. The two end panels of the bottom chords also should generally be laced. This is more important in railway bridges where the thrust from braked trains, either steam or electric, may be considerable; and in this case they should not be omitted. In ordinary highway spans, however, such a refinement is not necessary, and batten-plates ordinarily can be adopted throughout the bottom chord. Batten-plates 9″ × ⅜″ spaced three (3) feet centres will suffice for ordinary sections; but for the larger ones, the batten dimensions will have to be determined according to the specifications in Chapter LXXVIII. For the very heavy sections lacing will have to be resorted to from the view-points of both fabrication and shipment as well as from that of erection.

Stay-plates must be used at the ends of all built members, and they should conform to the specifications of Chapter LXXVIII. They should be placed as near the ends of the members as practicable, and should be extended well inside of the gusset-plates. Where the latter are placed inside of the member it is difficult to obtain this result, and it is largely for this reason that the author advocates turning the angles inward. In the case of the top chords with the angles turned out, the proper staying of the web members likewise stays the bottom flange of the chord. In placing the stay-plates, care should be taken to see that all field rivets can be readily driven or that means can be arranged for so doing. In the lower chords the bottom lateral plates are extended across the member and form the lower stay-plates; and at the top two stay-plates are generally employed, one on each side of the panel-point. They should be arranged so as not to interfere with the driving of field rivets in the bottom chord. The stay-plates need not be placed at the extreme end of the posts near the floor-beams because the diaphragms effect the same purpose. They should, however, extend past the ends of the latter from six (6) to twelve (12) inches.

Diaphragms are required in the truss members at the panel-points for the full depth of the floor-beams so as to equalize the panel load on the two leaves of the truss. Where the bottom chords have their angles turned in, it is necessary to break the diaphragms and make them in two pieces. They are generally built of four angles and a web-plate. The outstanding legs of the angles must be of ample width to provide for the floor-beam connection. Similar diaphragms are necessary in the chords which carry the floor system in bridges that have shallow floors. When the floor-beams are placed close together, say from one (1) foot to two (2) feet centres, the diaphragms need not be located at every floor-beam, as a spacing of from three (3) to six (6) feet will suffice. When the beams are spaced much farther apart than two (2) feet, a diaphragm should be placed at every beam. The details must be ample to make the chord act as a whole.

A common but very faulty practice in detailing trusses with single-plane sway-frames and portals is to connect these directly to the webs of the inner channels of the web members without the use of cross diaphragms. The latter detail should never be omitted. In fact, horizontal diaphragms to carry to the lacing at the sides of the posts any stresses that might exist in the bracing would be a further step in the right direction, but it is questionable whether such an innovation is of sufficient importance to be warranted.

Where the truss members are very heavy and thirty-six (36) inches or over in depth, cross-diaphragms, spaced from six (6) to ten (10) feet centres, should be employed throughout their length. Without the use of such diaphragms it is very difficult, if not impossible, to hold the members true to shape and line and to prevent their warping during fabrication. They likewise aid in securing accuracy in the widths of the members, which is a very important matter in large trusses where a failure in this particular would cause a large amount of extra shopwork. In addition, they help to stiffen the sections and to make the handling of the members, both in shipment and in erection, much easier and safer. For the smaller of the sections needing diaphragms, a three-eighths ($\frac{3}{8}$) inch web-plate with two angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, one on each end, will be sufficient, unless the member is very wide, in which case it will be necessary to add a stiffening angle at top and bottom on the other side of the plate.

The end angles should be placed against the webs of the truss members between the flange angles thereof, and the diaphragm itself need not be deeper than the length of its attaching angles, especially when the open faces are laced. When batten-plates are employed instead of lacing, the diaphragms should be placed at the points where these occur; and when their width demands horizontal stiffening angles, the diaphragm plate should extend up to the batten-plate, and the two should be connected by the said stiffening angles. In the heaviest sections it may be advisable to use eight angles for each diaphragm. In case the latter is extended over the flange angles of the chord, fillers should be inserted under the ends, unless side-plates already occupy the space. To secure accurate adjustment in the widths of the truss members, either the ends of the diaphragms are planed or the angles are set to a gauge made for the purpose.

The splicing of the members of a truss is one of the most important parts, if not the most important part, of the work of preparing the plans for a bridge; for if this is not done thoroughly and scientifically, the greatest care in the determination of the layout, the stresses, and the sections will be of little avail. The old saying "a chain is no stronger than its weakest link" cannot be emphasized too strongly in this connection. And it is a fact that while the sections of the members can be determined to a nicety, the splices and the connections (which, in reality, are nothing more than splices) do not lend themselves to definite analysis. Various assumptions, more or less arbitrary and uncertain, have to be made; and even so far as these are concerned, very little has been written that will throw any light on the subject. Of course, such statements do not refer to small sections, where splices of the simplest nature give practically no trouble whatsoever; but they do refer to the large structures that require very heavy sections and extraordinary details. No doubt this condition is due to the very recent development in large bridgework, heavy riveted trusses having come into use practically within the last ten years; consequently sufficient time has not yet elapsed for arriving at the best methods to be followed either from a theoretical standpoint or from one that is based on actual tests. While all engineers are familiar with the action of a beam with parallel edges and can test a section normal to these, the determination of the stresses on an oblique section presents a different problem; and when the edges are inclined to each other, the case becomes even more involved. All of these situations are met with in the testing of gusset-plates; and while they are of little importance in small spans where the gusset-plates are of minimum thickness and are more than ample for the stresses imposed on them, in heavy structures their consideration is most essential. In this chapter the author will give such methods as he has developed in the solution of their designing. They are the best that his office has been able to evolve, and can be safely employed until further investigations give simpler and more accurate methods.

The term " splice " is ordinarily assumed to refer to the connection between two parts of a member, the sections of which may be identical or may vary slightly but are of the same general outline. Each chord in this connection is considered as a single member, which in reality it is. This is true even in the broken top chord, the only difference being the necessity of making the splices at the panel-points. While the attachment of the web members to the chords (or, more strictly speaking, to the gusset-plates) is in the true sense of the word a splice, it is invariably spoken of as a connection, and will be so treated in this discussion when dealing with the gussets at the joints.

In trusses two types of splices are employed—the butt-splice and the lap-splice. The former is almost invariably used for compression members, as it is more economical because of the fact that the majority of specifications require the full butt-splice to be made only partly as strong as the main section, whereas the lap-splice has to be as strong as the member itself, or even stronger. It is evident that the butt-splice may require a greater total thickness of metal than the lap-splice; and the consideration of this fact may make it advisable to use the latter in very heavy trusses, in order to comply with the restrictions of the specifications concerning maximum length of rivet grip. In small tension members the butt-splice likewise is employed; however, when the sections are built up so that lap-splices can be used, they will be found the most economic as well as the most satisfactory from the designer's standpoint.

There are other considerations, though, that must be taken into account in deciding upon the style of splice best suited to any particular case, and these must be looked into carefully. The question of erection is one of the most important of them; and it is for this reason that the engineer in charge of the preparation of plans should be familiar with the methods that are likely to be adopted for the fieldwork. Telescoping connections of any kind should be avoided as far as possible, as it is difficult to place a member when it is necessary to shove it in from the end. This method is likewise dangerous, and it often causes delays that might prove serious. These difficulties do not present themselves in the butt-splice; and the lap-splice can be so arranged that this objection will not exist. The various parts can be cut so that the member to be erected can be lowered into place from above. It is true that it is necessary to drop certain plates over others, but with proper clearance no trouble will ensue from this source. In this connection it is well to call attention to the necessity of providing ample clearances for erection and to that of detailing the joints so that the members can be erected without having to be forced into place.

The number and the location of the splices depend on various conditions. The number of splices, naturally, should be made the minimum possible consistent with economy of material as well as with ease in handling, shipping, and erection. Field splices depend entirely upon this last consideration. In the first place, any section of a member must be neither too long nor too heavy for handling or for shipment. It must likewise be of such proportions that it can be properly loaded on the cars and that it will not encroach on the clearances specified by the various railroads over which the material will pass. For the smaller trusses the length for handling is the determining factor. As a rule, a length of about sixty (60) feet, or approximately a little over two ordinary panel lengths, should not be exceeded. If this limit is increased to any extent, it will be necessary to provide several points of support for the hoisting apparatus. Moreover, in the smaller spans there is no great saving of metal in omitting a few splices.

In the heavier trusses the weight of the member is the governing factor. Very heavy sections require special erection apparatus, and the facility with which this can be secured and the advisability of its use will depend on the location of the structure, consequently no fixed rules for splice location can be laid down. One of the heaviest single pieces on record for a simple truss bridge was that of the inclined end post of the 423′ riveted through-span of the Fratt Bridge, which piece weighed approximately one hundred and four (104) tons. This, of course, was an exceptional case; and ordinarily the pieces weigh much less.

In the case of a curved top chord, a field splice should be placed at every panel-point where there is a change in the inclination. When a straight piece extends over several panels, intermediate field splices may be necessary.

When a truss is erected by the cantilever method, it may be obligatory, or at least advisable, to put a field splice in each panel of each chord. This will depend on the weight of the member and on the nature of the erection equipment.

The use of the shop splice will depend on the make-up of the member between the field connections. Such a splice is necessary only in case a change occurs in the section between the said field connections. If the member extends over several panels, the change in stress will call for a change in section. As has been stated previously, it may be more economical to use the heavier section throughout and thus avoid the splice. Such a procedure will depend entirely upon the question of its economy; and in its solution will have to be duly considered the costs of material, labor, freight, and erection. In case a shop splice is considered advisable, as few of the parts of the member as possible should be spliced. If the smaller section can be used for the larger with the addition of a plate, this method should by all means be adopted. The additional plate can be connected directly to the gusset-plate, thus avoiding the usual splice. It is this fact that makes the adoption of the narrow plates between flange angles so advantageous in detailing, as is well illustrated in the bottom chord of the truss shown in Fig. 22*eee*. The conditions mentioned earlier in this chapter in regard to this detail must not be overlooked, however.

In the very heavy members, not only the change in the stresses but also the maximum procurable lengths of the component parts will determine the number of shop splices. These lengths will depend on the weights of the rolled sections, and it is consequently necessary to have at hand a list of the maximum lengths rolled for all the sections given in the manufacturers' handbooks.

The location of the chord splices will generally be governed by the erection of the structure. If possible, they should be placed just outside of the joints rather than at the panel-points, as such splices are easier to arrange for as well as to design. Moreover, they accommodate themselves better to the erection of the trusses. The most economical splice of this kind is the one in which the heavier section is sufficiently extended into the panel of the lighter member to permit the splicing of the latter to the former. This is invariably done in shop splices, and also in field splices when the trusses are erected on falsework. For cantilever erection, however, it is necessary to have the splice just ahead of the joint so that each panel can be placed complete before the traveler is run out to erect the next panel. As stated previously, inclined top chords and chords with unusually large sections must be spliced in the joints. This is such an important point as to warrant its reiteration.

Tension splices, and compression splices in which bearing is not relied upon, should be figured for the full strength of the member. In very heavy members where lap-splices are employed they should be made stronger than the main section on account of the uncertainty in stress distribution. The specifications in Chapter LXXVIII require ten (10) per cent excess of strength in such cases. Where the section is milled for a full bearing, a certain reliance can be placed on the abutting surfaces for transmitting the stress between the parts of a member. The author assumes the bearing good for forty (40) per cent of the actual section, and designs the splice-plates for the remaining sixty (60) per cent. All parts of each member should be fully spliced so that every section taken is up to full strength. The angles are generally neglected in this respect, the detailer assuming that the stress from these in some manner or other will travel across the cut. Such a defect in design shows either neglect or inexperience on the part of the detailer. Each leg of the angle should be spliced directly, although it is not always convenient to do this. It may be more advantageous to take the greater part of the stress from the angle through one leg, as is done in the flange of a plate-girder. In any case, however, there must be enough rivets connecting the angles to the splice-plates to develop their capacity fully. The use of the lighter sections of angles is advantageous in this regard, as fewer rivets are required for the development of their full strength. Deep chords carrying direct loads, and consequently acting as beams, should also have each individual part fully spliced, even though each part may not be fully stressed. It might be permissible under certain circumstances to splice such sections for the stresses only; but the author does not approve of this practice and advises against it as a general rule.

The splice-plates should be placed on each side of each part of the section so that the centre line of stress throughout the connection will be practically straight or will deviate from a straight line connecting the centres of the two members as little as possible. This arrangement gives a well-balanced detail; and it can be effected in most cases. An important advantage involved thereby is the fact that it brings the rivets into double shear, thus requiring a smaller number of them than would otherwise be needed. In small members made up of minimum sections of either rolled or built channels, it is uneconomical to adopt this plan; and, consequently, a single plate on one side of each leaf will suffice. However, where the before-mentioned arrangement can be employed without undue waste, it should by all means be resorted to.

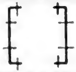
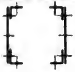
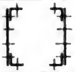
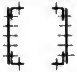
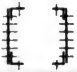
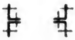
For butt-splices the rivets developing the plates are figured in single shear, double shear, or bearing, depending on whether one or more splice-plates are used and upon the thickness of the metal in which the strength is to be developed. In this case the web-plates are assumed to act as a unit with planes of shear on each outside surface thereof. Lap-splices, on the other hand, cannot be figured so simply, as the various plates and angles are cut at different points and spliced separately. Under this condition, the splice-plates on one side may be considerably closer to the plate cut than on the other side, which causes a greater amount of stress to go to the nearer plate. In figuring such splices it is best to assume that the stress from any plate is divided between the splice-plates on the two sides in inverse proportion to their distances from the said plate. While this may not be strictly true, it is the best approximation thus far suggested. Evidently, when the web-plates lap over each other, they must be considered as splice-plates and treated accordingly.

In any splice the more closely together the rivets are placed, the more compact, and consequently the more economical, will the said splice be. In compression splices the only limitation is the minimum distance from centre to centre of rivets required by the specifications. In tension splices, on the other hand, it is necessary in addition to watch the net section of the member. This is generally determined at the first line of rivets in the splice. It would be extremely uneconomical to reduce the section by a full row of holes having the minimum spacing allowed; consequently, a number of rivets sufficient to tack the plates properly together as well as to start developing their strength should be used at this place. As a guide to what the author considers a proper number of holes to take out of tension members of various depths so as to determine the net section, Table 22*b* is given. The holes are indicated by the cross-lines. After the first line of rivets is passed, no additional holes can be taken out unless there are sufficient rivets in front of the section in question to compensate for them. Naturally, it is desirable to increase the number of rivets in any single row as rapidly as possible until the full number is used; and it is for this reason that the maximum allowable spacing of twelve (12) inches is seldom employed in the first row. This is especially important in thick sections where each hole out requires several rivets to compensate for it, but in members of thin or minimum sections it is not so essential.

As previously stated, the author designs lap-splices so that they shall be ten (10) per cent stronger than the member spliced. In order to get this extra strength into the splice-plates as soon as possible, it is developed by both the rivets and the splice-plates before the first plate is cut. The strength of the said plate is developed at the same time.

TABLE 22*b*.

NUMBER OF RIVET HOLES TO BE DEDUCTED FROM BUILT-UP TENSION MEMBERS.

Section	Width of Plate	Holes out of Each Plate or [Web	Holes out of Each Angle or [Flange	Arrangement of Rivets
Rolled Channels		2	1	
Built Channels	Up to 18"	3	2	
	19" to 32"	4	2*	
	33" to 48"	5	2*	
	49" to 60"	6	2*	
Four Angle I			2	

* This figure may be increased to 3 by lacing

To show clearly the method employed in figuring a tension lap-splice, the following example is given. This splice was used in the bottom chord of the 271′ riveted truss span mentioned previously and shown in Fig. 22*eee*. This structure was designed under the specifications of the American Railway Engineering Association. The rivet values as given therein are 12,000 pounds and 10,000 pounds per square inch for shear on shop and field rivets, respectively, and twice these values for bearing; and the unit tensile stress in the steel is limited to 16,000 pounds. The members spliced are the bottom chord sections L_3L_4 and L_4L_4'. The latter is extended into panel L_3L_4, and, consequently, the splice is figured for the smaller section. Fig. 22*ggg* gives the details of the completed splice for the whole section in addition to the make-up of each member; while Fig. 22*hhh* shows the complete figures for one leaf of the splice.

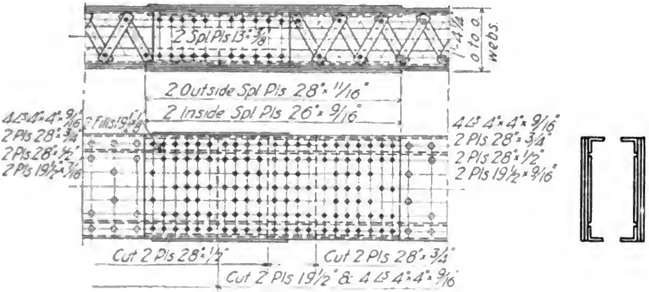


FIG. 22*ggg*. Tension Lap-splice in Panel L_3L_4 of the 271-foot Spans of the Great Northern Railway Co.'s Bridge over the Yellowstone River.

As is seen from Fig. 22*eee*, the chord sections are well arranged for lap-splicing; and in this particular splice the only difference in the two members is in the small plates between the angles. In L_4L_4' these are of the same thickness as the angles, which condition precludes the necessity of fillers on this side of the point where these plates and the angles are cut. In L_3L_4 the thickness of these plates is slightly less than that of the angles, calling for the ⅛" filler shown.

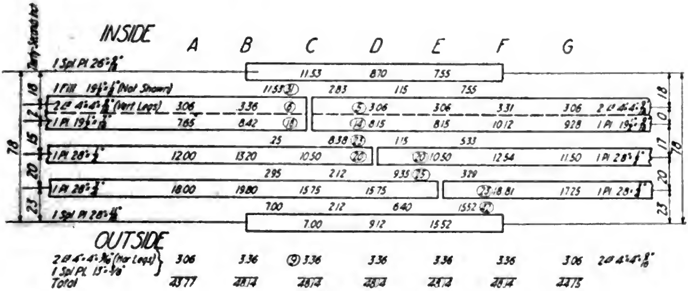

INSIDE
A
B
C
D
E
F
G
1 Spl Pl 26"×9/16"
1153 870 755
1 Fill 19½"×1/8" (Not Shown)
1153 ㉛ 283 115 755
2 ∠s 4"×4"×9/16" (Vert Legs) 306 336 ⑥
⑤ 306 306 331 306 2 ∠s 4"×4"×9/16"
1 Pl 19½"×7/16" 765 842 ⑱
⑭ 815 815 1012 928 1 Pl 19½"×9/16"
25 838 ㉓ 115 533
1 Pl 28"×1/2" 1200 1320 1050 ⑳
⑳ 1050 1254 1150 1 Pl 28"×1/2"
295 212 935 ㉕ 329
1 Pl 28"×3/4" 1800 1980 1575 1575
㉓ 1881 1725 1 Pl 28"×3/4"
700 212 640 1552 ㉗
1 Spl Pl 28"×11/16"
700 912 1552
OUTSIDE
2 ∠s 4"×4"×9/16" (Hor Legs)
1 Spl Pl 13"×3/8"
306 336 ⑨ 336 336 336 336 306 2 ∠s 4"×4"×9/16"
Total 2377 2814 2814 2814 2814 2814 2275
78 18 12 15 20 23
18 0 17 20 23 78

FIG. 22*hhh*. Diagram Illustrating the Method of Figuring a Tension Lap-splice.

The first thing to arrange is the cutting of the plates and angles.

As shown in Fig. 22*hhh*, the angles and the plates between them are cut at *C*. It is necessary to cut these plates at the same point as the angles so as to permit one member to be dropped down over the other. The adjacent plates are cut at *D*, and the outside ones at *F*. The arrangement of cutting the outside plates at *D* and the adjacent ones at *E* will not give a splice as satisfactory for erection as the one shown, on account of the tongue-and-groove joint produced thereby in each leaf. The diagram in Fig. 22*hhh* is laid out as shown for convenience in following the details of the splice. The horizontal legs of the angles are spliced with top and bottom plates; while the vertical legs thereof and the web-plates are spliced with vertical plates inside and out. For this reason the figures for the horizontal legs of the angles are separated from the rest as shown at the bottom of the diagram. The plates are sketched with sufficient widths to take the necessary figures, while they are separated by a space in which to indicate the stresses passing across the planes between adjacent plates. The angles and the narrow plates that lie between them are separated by a dotted line to indicate that they are in the same plane, and that consequently no rivets can pass between them. After the plates are laid out, their distances from centre to centre are then given at each end for each member in thirty-seconds of an inch. These are used for determining the lever arms for moments. At the start it is essential to assume the thickness of the splice-plates approximately in order to fix their distances from the adjacent plates; and if they are changed materially in the design, it will be necessary to run through the figuring of the splice a second time. This also may be obligatory in checking the splice from both ends, as will be explained later. Next, the make-up of the sections is given, each part thereof in its proper place; and at *A* and *G* the net areas of the latter are indicated. As the splice is figured for the smaller member, it is possible to enter the splice at the right with a greater number of rivets than at the left; however, the net area must not be reduced below that of the smaller section. As shown in Fig. 22*eee*, four (4) holes are taken out in the first vertical row for L_3L_4 and five (5) for L_4L_4'. At *B* are given the net sections of the various parts of the member L_3L_4, increased by ten (10) per cent; and at *F* are recorded the net sections of the various parts of the member L_4L_4', increased sufficiently to make the total section here equal that at *B*. The splice will be figured to develop these values. Instead of figuring with stresses, the splices are carried through on the basis of square inches of section which work at the normal unit stresses. This is convenient from the standpoint of space as well as from that of clearness. These areas are readily convertible into stresses or rivets by the preparation of a table giving the value of one square inch of metal in pounds, as well as in rivets for single shear and bearing on various thicknesses of plates for both shop and field rivets. For this particular splice these values are as follows:

1 square inch of metal = 16,000 lbs.
= 2.66 7/8″ rivets, field, single-shear
= 1.33 “ “ “ double “
= 2.09 “ “ “ bearing on 7/16″ plate
= 1.83 “ “ “ “ “ 1/2″ “
= 1.62 “ “ “ “ “ 9/16″ “
= 1.34 “ “ “ “ “ 11/16″ “
= 1.21 “ “ “ “ “ 3/4″ “

As stated before, the net section must be taken care of first in order to secure the maximum number of rivets in a single row as soon as possible. In L_3L_4 one hole out requires 2.3 field rivets in double shear to compensate for it, while in L_4L_4' 2.5 rivets are required; consequently, one more rivet could be added to the second row in L_3L_4 and two in L_4L_4'. (See Fig. 22*ggg*.) The third row in L_3L_4 could be filled completely. As the number of vertical rows of rivets cannot be reduced by doing this, it would simply be a waste of money to put in these extra rivets, for the reason that they are not needed. In addition to the net section the excess section must be transferred to the splice-plates as soon as possible, and, consequently, it is developed in conjunction with the first plate to be spliced before the said plate is cut. Therefore at *C* and *E* the other plates have a value equal to their actual minimum net section alone, and this is maintained until each plate is cut.

In determining the value of either splice-plate, moments are taken at *C*, *D*, and *E* about the other plate as a centre for the areas to be transferred from the various parts of the member to these plates. This moment is divided by the distance from centre to centre of plates, and the resultant is added algebraically to the value of the splice-plate at the previous section in order to find its value at the point in question. The determination of the values of these plates at the said points is as follows, figuring from the left:

Inside splice-plate at *C*,

$$0 + \frac{1}{78}[(3.36 \times 60) + (8.42 \times 58) + (2.70 \times 43) + (4.05 \times 23)] = 11.53.$$

Outside splice-plate at *C*,

$$3.36 + 8.42 + 2.70 + 4.05 - 11.53 = 7.0.$$

Inside splice-plate at *D*,

$$11.53 + \frac{1}{78}[(10.50 \times 43) - (3.06 \times 60) - (8.15 \times 60)] = 8.70.$$

Outside splice-plate at *D*,

$$7.00 + 11.53 + 10.50 - 3.06 - 8.15 - 8.70 = 9.12.$$

Inside splice-plate at *E*,

$$8.70 + \frac{1}{78}[(15.75 \times 23) - (10.50 \times 43)] = 7.55.$$

Outside splice-plate at *E*,

$$9.12 + 8.70 + 15.75 - 10.50 - 7.55 = 15.52.$$

As a check on section E we can figure the values of the plates from the right, obtaining

Inside splice-plate at E,

$$0 + \frac{1}{78}[(0.25 \times 60) + (1.97 \times 60) + (2.04 \times 43) + (18.81 \times 23)] = 8.38.$$

Outside splice-plate at E,

$$0.25 + 1.97 + 2.04 + 18.81 - 8.38 = 14.69.$$

This agreement is sufficiently close, the difference being due to the fact that the two chord sections are unlike. By adding up the areas at C, D, and E, they should be the same as at B and F. This is indicated at the bottom of the diagram.

As would be expected, the inside splice-plate requires the maximum area at C, while the outside plate requires it at E. A 26″ × 9/16″ plate with five (5) holes out has a net section of 11.80 square inches, which is all right for the inside plate. For the outside, a 28″ × 11/16″ plate with five (5) holes out and having a net area of 15.81 square inches will suffice. For the top and the bottom plates splicing the horizontal legs of the angles, a 13″ × 3/8″ plate having a net area of 4.12 square inches is used. These plates also act as stay-plates and must fulfil the specifications as to thickness—which they do. As will be noted by referring to Fig. 22*ggg*, the maximum number of rivets is not provided at the critical sections of the side splice-plates; for if this were done, the splice-plates would be increased in thickness without decreasing their lengths.

One hole out of the 11/16″ outside plate requires 1.9 field rivets in single shear to develop it, and for the 9/16″ inside plate 1.5 rivets are needed; consequently, the second row of rivets can be completely filled in each case.

To determine the number of rivets required between the various points B and C, C and D, D and E, and E and F, it is necessary to figure the areas passing from one plate to the other across the planes between them. Take the part of the splice between B and C. Starting at the top, the total value of the inside splice-plate (11.53 square inches) has to pass across the plane between it and the adjacent plate and angles. As the area of this plate and that of the angles equals 11.78 square inches, the difference between this and 11.53 square inches, equal to 0.25 square inches, has to cross the next plane in going to the outside splice-plate. In crossing the 28″ × 1/2″ plate it picks up 2.70 square inches, making 2.95 square inches to pass over the next plane. From the next plate 4.05 square inches are added, giving 7.00 square inches across the outside plane, which is equal to the value of the outside splice-plate at C. Comparing these various values, we see that the maximum section on any plane is 11.53 square inches adjacent to the inside splice-plate. Converting this into rivets by the use of the table previously given, we find that thirty-one (31) are needed in field single shear. The angles require six (6) field rivets in bearing and the 19½″ × 7/16″ plate calls for eighteen

(18). It is not necessary to consider the other plates and planes, as it can be seen by inspection that they will need fewer rivets than those just computed. By examining Fig. 22*ggg*, we find that there are six (6) rivets in the vertical legs of the angles, twenty (20) in the $19\frac{1}{2}'' \times \frac{7}{16}''$ plate, and thirty-two (32) in the splice-plates, which arrangement meets all the requirements of the calculations. In the same way the other rivets are determined. It must be remembered that, as the rivets distribute the stresses in inverse proportion to the lever arms, it is necessary to end the two splice-plates at the same points. An inexperienced designer might be tempted to save a little metal by shortening the splice-plates at the ends where the requirements are apparently small. The top and the bottom splice-plates need five (5) rivets in each angle, which the detail in Fig. 22*ggg* provides.

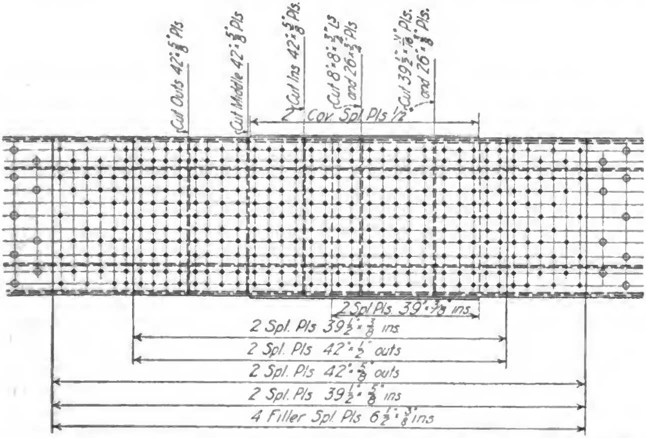


FIG. 22*iii*. Bottom Chord Tension Lap-splice for the 287-foot Span of the O.-W. R. R. & N. Company's Bridge over the Willamette River at Portland, Ore.

The figuring of splices for heavier members is carried out in the same manner as just explained for those of moderate sections. Two or more splice-plates may be needed both inside and out, and these must be economically arranged as to both thickness and length. To illustrate such a splice, Fig. 22*iii* is given. This is taken from the design of one of the 287′ riveted truss spans of the O.-W. R. R. & N. Co.'s Bridge over the Willamette River at Portland, Ore. The section of this member is given in Fig. 22*ccc*.

Splices at the panel-points are figured in the same manner as are those in the panel, except that the gusset-plates are used as splice material and must be dealt with accordingly. As this type of splice depends to such a great extent upon the gusset-plates, an example of it will not be given until after the gusset-plates have been discussed.

Compression splices are designed in the same manner as are tension splices, except that the net section does not have to be considered.

The gusset-plates, as previously stated, are needed to connect the web members of a truss to the chords. For the lighter trusses, two plates per joint are used, placed either inside or outside of the chords and web members. These gussets are really splice-plates, and the connections are computed in just the same way as previously described and exemplified. While two plates are sufficient for the smaller spans, four or more are needed for the heavier ones. These plates are either treated as the single plates already mentioned or are spliced-in with the various parts of the web and chord sections.

The method of shingling-in the truss members with the gussets will depend upon the amounts of the stresses to be transmitted. If the stresses are large and the rivets are thrown into single shear, the connections become excessively long and the plates unnecessarily large. With the shingling of the sections the rivets are thrown into bearing and double shear, materially reducing both the number of rivets and the sizes of gussets.

On account of the irregularity of the joints, it is necessary to mention a few special points that need attention. In determining the end cuts of the various parts of a member, it should be kept in mind that beveled cuts are objectionable. Of course, it is not always advisable to avoid them, especially in the web-plates; but there is little excuse for cutting the angles on a bevel. Beams and channels with skewed ends have to be trimmed by sawing, which is more expensive than shearing. Lug angles likewise should be avoided, when possible; however, it is frequently advantageous to use them in order to develop the angles within the gusset-plates as otherwise longer gussets would be needed. The rivets in any connection should be balanced about the centre-line of the member. The gusset-plates should have as few cuts as possible; plates with parallel edges are to be preferred to those with the edges inclined. Making the tops of the bottom-chord gusset-plates in through bridges horizontal and lining them up add materially to the appearance of the span; but too much expense should not be entailed in so doing. The gusset-plates should be riveted to those members which will involve the easiest erection. As a rule, the top plates are riveted to the upper chords, the end bottom plates to the end posts, and the other bottom gusset-plates to the lower chords. Care should be taken to see that both the shop riveting and the field riveting are arranged so that the plates will not interfere with the erection.

In detailing a joint it is first necessary to lay out, tentatively, the members and the rivets in the various connections.

Figs. 22*jjj* and 22*kkk* will prove very helpful in determining the number of rivets for any stress from zero to 100,000 pounds for rivets in shear and in bearing. For larger stresses the rivets can be determined by proportion. The gusset-plates are then tested for tearing out, crushing, bending, and direct stresses. The tearing out or the crushing of the gussets, due to the web connections, will depend on the shape of the gusset and the type of splice. It is, therefore, necessary to test the plates around the periphery of the rivet group as well as along the sides and on any intermediate row. Also, sections through the rivets carried over to the edges of the gussets must be considered. When the rivets are in single shear only, the gussets are almost invariably safe. One or two tests, however, will make the fact certain. When the rivets through the

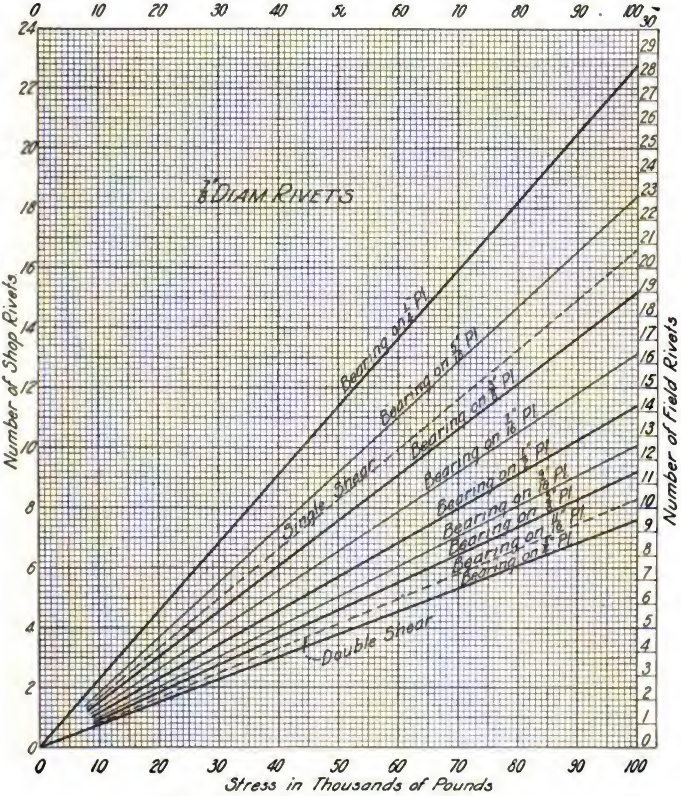


FIG. 22*jjj*. Rivet Diagram for 7/8" Rivets.

gussets are in bearing, numerous sections must be tested to prevent overstress. The outline of the different plates at any joint, when more than two of them are used, will depend on this condition. The sections are

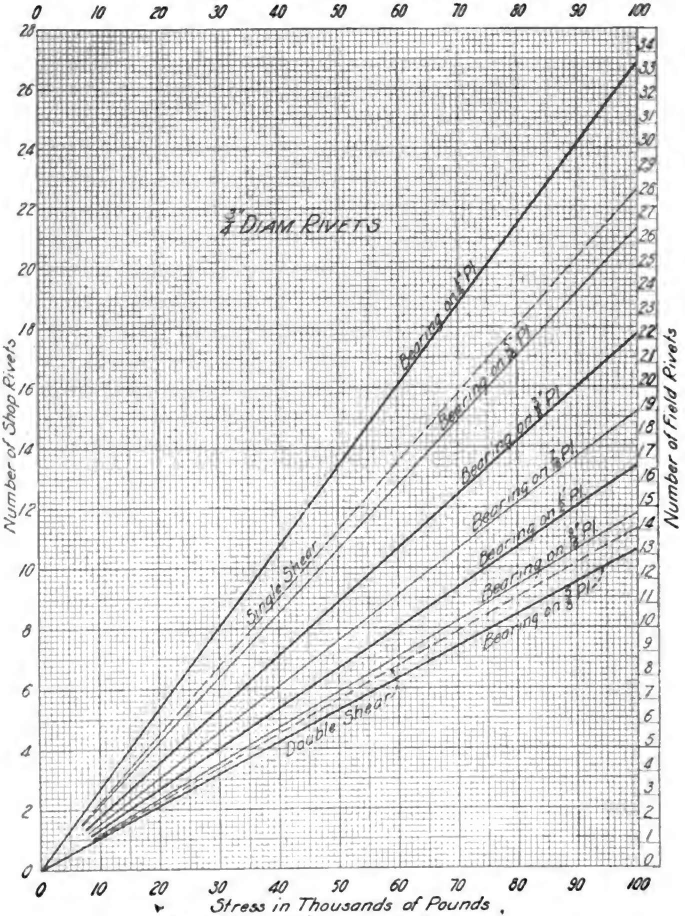


FIG. 22kkk. Rivet Diagram for $\frac{3}{4}$" Rivets.

figured for shear on surfaces parallel to the member and for tension or compression on those that are normal thereto.

Testing the gusset-plates for direct and bending stresses presents a somewhat uncertain problem on account of the irregularity in the plates, in the sizes of the rivet groups, and in the various arrangements thereof. The sections should be taken parallel to one of the intersecting members; and the various forces should be resolved normal and parallel to the sections. Taking the layout shown in Fig. 22*lll* with the web members below the chord and the connecting rivets uniformly distributed in the distance ka, we find the point of maximum moment for a section parallel to the chord to be, in general, at a distance $\frac{a}{2}$ below its centre line, and the moment to be

$$M = \frac{a}{4k} S_D \sin \alpha,$$

in which S_D is the stress in the diagonal, and α is the angle between the post and the diagonal. When k is less than ½, the maximum moment occurs at the end of the diagonal, and is

$$M = a(1 - k)S_D \sin \alpha.$$

For values of k up to 0.65, the maximum moment will not exceed the moment at the end of the member more than ten (10) per cent, and consequently for such a condition the moment at the end of the connection will suffice; otherwise the point of maximum moment should be used. The direct stress on the section is zero, since the vertical component of S_D is equal to the stress in the post; and the shear is equal to the horizontal component of S_D.

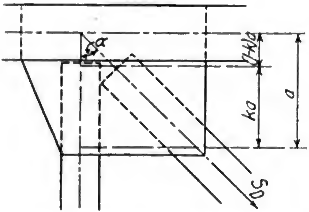


FIG. 22*lll*. Gusset-Plate Diagram

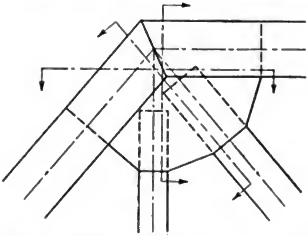


FIG. 22*mmm*. Sections for Testing Gusset-Plates at the Hip Joint.

While the layout shown in Fig. 22*lll* is the ideal joint, it occurs more often as the exception rather than the rule. Sometimes the post extends into the chord, in which case the point of maximum moment is nearer the centre line of the latter than $a/2$. Again, the post connection may extend below that of the diagonal, in which case the point of maximum moment is approximately $\frac{a + b}{4}$ below the centre line of chord, in which expression b is the depth of the post connection below the said centre line. In case the web members are shingled into the gussets and the rivets through them are thrown into bearing for a part of the connection, this fact must be taken into account by making *a* greater than the distance to the end of the gusset-plate. In case the gusset-plates help to splice the chord, the stresses induced in them by this action must also be used in figuring the moments and the direct stresses on any section normal to the said chord. Sections parallel to the posts are tested in the same manner as those parallel to the chords. The sections indicated in Fig. 22*mmm* should be used in testing the hip joint. The extreme fibre stresses on inclined edges must be multiplied by the square of the secant of the angle between the edge and a normal to the section being tested in order to determine the true stress.

To illustrate clearly the methods just described, the figuring of the gusset-plates at the hip of the span shown in Fig. 22*eee* will now be explained. Fig. 22*nnn* gives a sketch of this joint, showing the sections to be tested and the loads for one leaf of each of the members meeting there. The unit stress in tension is 16,000 pounds per square inch, in compression on the top chord 13,400 pounds, and in compression on the end post 10,000 pounds, although the allowable intensity for this last member is 11,940 pounds. The numbers of rivets required for the different parts of the various members are as follows:

Member	Part	Area (Sq. Ins.)	Unit Stress, Pounds per Sq. In.	Rivets Required
U_1L_0	1 cover-plate 28″×⅝″	17.50	10,000	29 field single shear
	2 L 4″×4″×½″	7.50	10,000	13 “ “ “
	2 L 6″×6″×¾″	16.88	10,000	28 “ “ “
	2 webs 26″×¾″	39.00	10,000	65 “ “ “
	2 webs 15½″×¾″	23.25	10,000	39 “ “ “
	2 plates 10″×11/16″	13.75	10,000	23 “ “ “
U_1U_2	1 cover-plate 28″×⅝″	17.50	13,400	33 shop single shear
	2 L 4″×4″×½″	7.50	13,400	14 “ “ “
	2 L 6″×6″×¾″	16.88	13,400	32 “ “ “
	2 webs 26″×¾″	39.00	13,400	73 “ “ “
	2 plates 6″×9/16″	6.75	13,400	13 “ “ “
U_1L_2	4 L 4″×4″×½″	11.00	16,000	30 field single shear
	2 plates 24″×¾″	30.00	16,000	80 “ “ “
U_1L_1	4 L 6″×4″×9/16″	16.74	16,000	44 “ “ “
Splice plates	2 plates 15½″×¾″	22.75	16,000	62 “ “ “
	2 plates 34″×½″	34.00	16,000	91 “ “ “

As laid out, the top angles of the end post are developed directly into the gusset-plates; and in addition they transfer the stress from the cover-plate, in conjunction with the connection angles on the inside. Forty-two (42) field rivets in single shear are required for the top angles and the cover-plate, and forty-six (46) are provided. Forty (40) rivets connect the 15½″ × ¾″ plates, whereas thirty-nine (39) are needed. The bottom angles require twenty-eight (28) rivets, where twenty-six (26) are shown; while the 10″ × $^{11}/_{16}$″ plates call for twenty-three (23), with twenty-four (24) provided. The 26″ × $^{3}/_{4}$″ webs have one hundred and twenty (120) rivets, while sixty-five (65) are shown to be necessary by the calculations. Although this is more than are needed, the rivets cannot well be reduced in number on account of the other details. In the same way the various parts of U_1U_2 will be found to be amply developed. In this case it was necessary to use lug angles to help transfer the stress from the bottom angles. The equivalent of one hundred and four (104) shop rivets in single shear is provided in the web, whereas only seventy-three (73) are needed. Hanger U_1L_1 is connected with forty-four (44) rivets, all of which are required; while U_1L_2 calls for one hundred and ten (110) rivets with one hundred and twelve (112), as detailed. The net sections of both members are taken care of as shown.

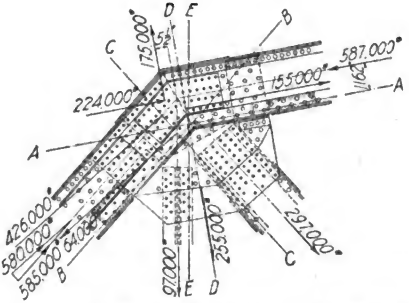


FIG. 22*nnn*. Hip Joint for the 271-foot Spans of the Great Northern Railway Co.'s Bridge over the Yellowstone River.

It might be noted that the rivets through the outside splice-plates do not really act at full efficiency in taking stress from the end post and chord, since these plates can take loads perpendicular to the cut of the web-plates only; but, on the other hand, the bearing resistance of the milled surfaces is neglected, so that it is entirely satisfactory to consider the rivets in the said outside plates to be of full efficiency.

We shall now test the various sections marked *AA*, *BB*, etc. Acting on section *AA*, we have a force of 587,000 pounds from U_1U_2 along its centre line; and from L_0U_1 one of 224,000 pounds in the opposite direction in addition to a force of 175,000 pounds normal to the section and located five and one-half (5½) inches to the left of the panel point. These last two forces are the components of that part of the stress from L_0U_1 acting above the section, viz., $\frac{62}{128}$ of the total stress. This ratio was found by counting the number of rivets in the entire connection through the upper end of the inclined end post, and that portion thereof lying above the section *AA*. These forces, as well as all others to be considered, are taken for one side only. They are indicated in Fig. 22*nnn*. The moment of these forces about the centre of gravity of the section is

$$M = 363{,}000 \times 11.62 - 175{,}000 \times 7 = 3{,}000{,}000 \text{ in. lbs.}$$

The gross area of the section is 72″ × 1″ = 72 sq. inches. Therefore the bending stress is

$$f_b = \frac{6 \times 3{,}000{,}000}{1 \times 72 \times 72 \times .63 \times .63} = \pm 8{,}800 \text{ lbs.},$$

the factor 0.63 in the denominator being the cosine of the angle between a normal to the section and the inclined edge of the plate; and the direct stress is

$$f_d = \frac{175{,}000}{72} = +\ 2{,}500 \text{ lbs.},$$

giving extreme fibre stresses of $f_c = 6{,}300$ lbs. and $f_t = 11{,}300$ lbs. The unit tensile stress on the net section will evidently be larger than 11,300 pounds per square inch by about twenty (20) per cent, since there are rivets six (6) inches between centres in the tension portion, or say 13,600 pounds per square inch. The unit shear on the section is

$$S = \frac{363{,}000}{72} = 5{,}000 \text{ lbs.}$$

These stresses are all satisfactory. The stresses on section *BB* will be found to be about the same as for section *AA*.

On section *CC*, we have acting normal to the section, from the end post,

$$C_P = \frac{94}{128} \times 580{,}000 = 426{,}000 \text{ lbs.},$$

and from the hanger,

$$T_H = 64{,}000 \text{ lbs.}$$

For the section we have the following properties:

Section	Area (*A*)	Distance (*x*) from Centre Line of Chord to Centre of Gravity of Plate	Ax	h	Ah^2	I_c	Total I
1 plate 68″ × ½″....	34.0	22	748	7.7	2,020	13,100	....
1 plate 52″ × ½″....	26.0	14	364	0.3	0	5,860	....
1 plate 30″ × ½″....	15.0	3	45	11.3	1,920	1,120	....
*1 plate 15½″ × ⅜″..	5.8	0	0	14.3	1,190	120	....
Totals...........	80.8	14.3	1,157	....	5,130	20,200	25,330

* Rivets develop only this much of the 15½″ × ¾″ plate.

We then have

$$M = 426{,}000 \times 14.3 - 64{,}000 \times 4.3 = 5{,}815{,}000 \text{ in. lbs.},$$
$$C = 362{,}000 \text{ lbs.},$$
$$f_d = 362{,}000 \div 80.8 = -\ 4{,}480 \text{ lbs.},$$

$$f_b \text{ (comp.)} = \frac{5{,}815{,}000 \times 26.3}{25{,}330} = -\ 6{,}040 \text{ lbs.},$$

$$f_b \text{ (tens.)} = \frac{5{,}815{,}000 \times 41.7}{25{,}330} = +\ 9{,}590 \text{ lbs.},$$

$$f_c = 10{,}520 \text{ lbs.},$$

and
$$f_t = 5{,}110 \text{ lbs.}$$

Hence the section is strong enough, as the unit shear is low.

On section *DD*, we have from U_1U_2,

$$C_C = 587{,}000 \text{ lbs.},$$

and from U_1L_2,

$$T_D = 155{,}000 \text{ lbs.},$$

both of these forces being normal to the section. For the section we have the following properties:

Section	Area (A)	(x)	Ax	h	Ah^2	I_c	Total I
1 plate 68″×½″	34.0	22.0	748	8.7	2,560	13,100	....
1 plate 52″×½″	26.0	14.0	364	.7	10	5,860	....
1 plate 30″×½″	15.0	3.0	45	10.3	1,590	1,120	....
1 plate 15½″×¾″	11.6	0.0	0	13.3	2,060	230	....
Totals	86.6	13.3	1,157	....	6,220	20,310	26,530

We then have

$$M = 587{,}000 \times 13.3 - 155{,}000 \times 5.6 = 6{,}930{,}000 \text{ in. lbs.},$$

$$C = 432{,}000 \text{ lbs.},$$

$$f_d = 432{,}000 \div 86.6 = -\ 5{,}000 \text{ lbs.},$$

$$f_b \text{ (comp.)} = \frac{6{,}930{,}000 \times 25.3}{26{,}530} = -\ 6{,}600 \text{ lbs.},$$

$$f_b \text{ (tens.)} = \frac{6{,}930{,}000 \times 42.7}{26{,}530} = +\ 11{,}140 \text{ lbs.},$$

$$f_c = 11{,}600 \text{ lbs.},$$

and
$$f_t = 6{,}140 \text{ lbs.}$$

The shear on *DD* is

$$S = 255{,}000 \div 86.6 = 3{,}000 \text{ lbs. per sq. in.}$$

The values for f_c, both on *CC* and on *DD*, are lower than is necessary. However, it must be remembered that these figures are made on gross sections, and that they will be increased from ten (10) to fifteen (15) per cent on the net section. Section *EE* is practically the same as *DD*.

We shall now test the gusset-plates for tearing out by the diagonal. Figuring the section along the outer lines of rivets and across the third row, we have:

1 Plate 15″ × ½″ at 10,000 lbs. =	75,000 lbs.
1 Plate 15″ × ½″ at 16,000 lbs. =	120,000 lbs.
Total strength of section =	195,000 lbs.
Stress, 14 rivets at 6,000 lbs. =	84,000 lbs.

Each extra row of rivets adds 6 × 6,000 lbs., or 36,000 lbs., while the increase in the strength of the gusset-plate is 6 × ½ × 10,000 lbs., or 30,000 lbs. Even though each row of rivets is stronger by 6,000 pounds than the extra plate, they cannot overcome the big excess of the plate over the rivets (111,000 pounds) on the section first figured. Moreover, after the next two rows, another one-half (½) inch plate is added.

From the foregoing calculations we see that the joint is well designed in every respect. Joints at other points are figured in the same manner as are also joints with the members shingled into the gussets. To illustrate the latter, Figs. 22*ooo*, 22*ppp*, 22*qqq*, and 22*rrr* are given.

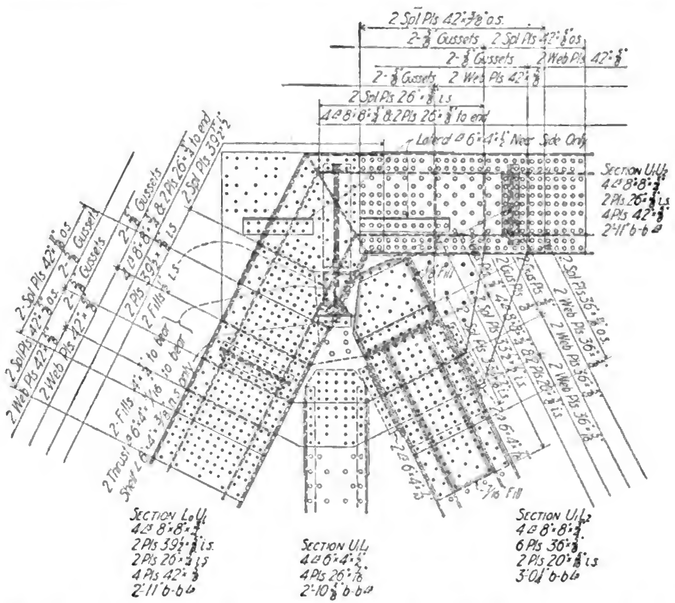


FIG. 22*ooo*. Hip Joint of the 287-foot Span of the O.-W. R. R. & N. Company's Bridge over the Willamette River at Portland, Ore.

In this connection the splicing of the section of the bottom chords where metal is cut out by the floor-beam rivets must not be overlooked. In deep chords for heavy trusses, it is very important to give this point careful consideration.

In pin-connected spans the truss members are similar to those for riveted spans, except that eye-bars are generally employed for the parts under tension only, and that usually all the web members are hinged at their connections. It was formerly customary to make all diagonals of eye-bars, and to employ adjustable counters in all panels in which the stresses reversed; but first-class modern practice requires the use of stiff diagonals to care for the reversing stresses. The bottom-chord members in the two end panels should be stiff; and they are occasionally made of built sections throughout, as such construction permits of a much more satisfactory attachment of the lower lateral diagonals. Again, in certain cases the connections of the vertical posts and the hangers have been made rigid, thus leaving the diagonals as the only members with hinged ends. The great economy, however, in the pin-connected construction lies in making all the purely tension members of eye-bars, excepting only those in the two (or four) panels at each end of the bottom chords.

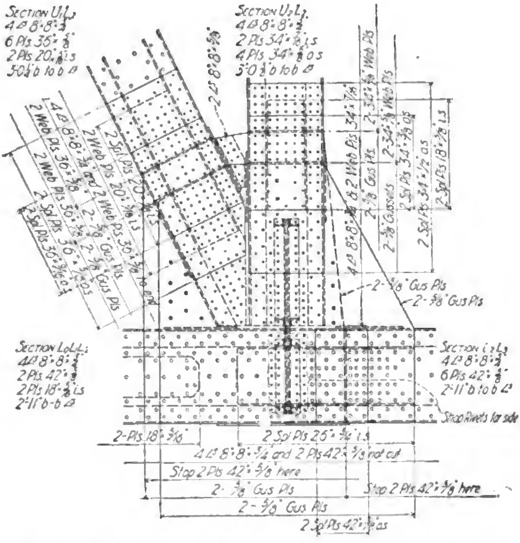


FIG. 22*ppp*. Bottom Chord Joint L_2 of the 287-foot Span of the O.-W. R. R. & N. Company's Bridge over the Willamette River at Portland, Ore.

It is evident that the packing at the joints affects the widths and arrangements of the truss members; hence care must be taken to see that neither the web members nor the chord members are made too narrow. The previous discussion on riveted trusses holds also for pin spans wherever applicable.

While the built members in general are arranged along the same outlines as those for riveted spans, for the chords of heavier trusses it is necessary to resort to three and sometimes to four main webs. This is especially advantageous when four or more bars are employed for the main diagonals, as it permits of an arrangement that prevents excessive bending stresses on the pins. Moreover, it reduces the thickness of the pin metal required for any one of the webs. Such construction is also necessary for the very large sections in long spans. Where it is not desirable to use more than two webs, in spite of the entailed necessity of either employing excessively large pins or else providing special details, the pin-diameters can be reduced by adding one or two short intermedi-

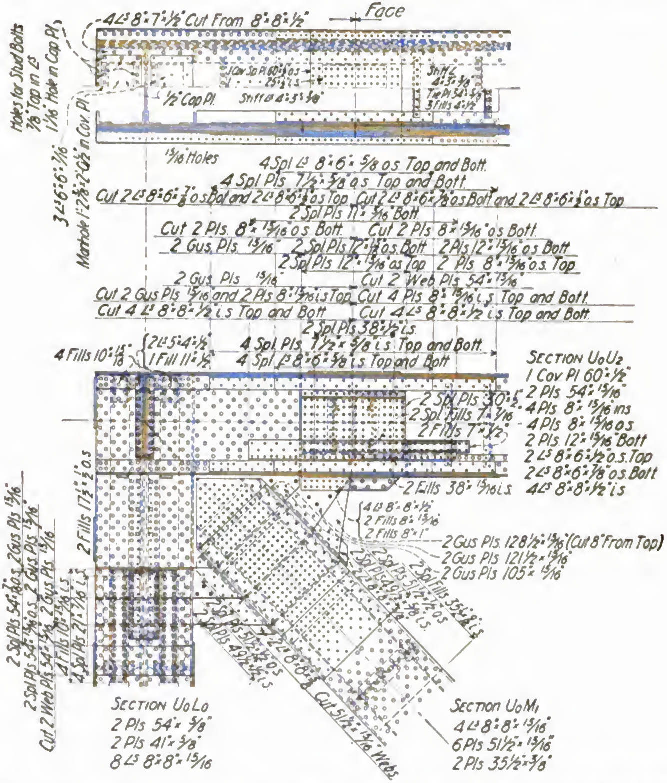


FIG. 22*qqq*. Top Chord Joint U_0 of the 423-foot Fixed Spans of the Fratt Bridge over the Missouri River at Kansas City, Mo.

ate webs at the ends of the members for pin bearing and connecting them to the main webs by cross diaphragms. Figs. 22*sss*, 22*ttt*, and 22*uuu* show three sections for large built members of pin-connected spans.

In determining the character and sizes of the various members, care should be taken to see that the pin-packing is satisfactory. In so doing, ample clearance must be allowed between the different members attached to each pin. For compound webs, one thirty-second ($^1/_{32}$) of an inch should be allowed for each additional plate above two on account of the tendency of such compound plates to build out or thicken. As far as practicable, rivets should not be countersunk, although they may be flattened; and the outstanding legs of angles should be cut just as little as possible, especially in compression members, as such members are materially weakened thereby. In any case projecting rivet heads and angle legs must be taken into account in determining the packing. Not less than one-sixteenth ($^1/_{16}$) of an inch should be allowed between adjacent

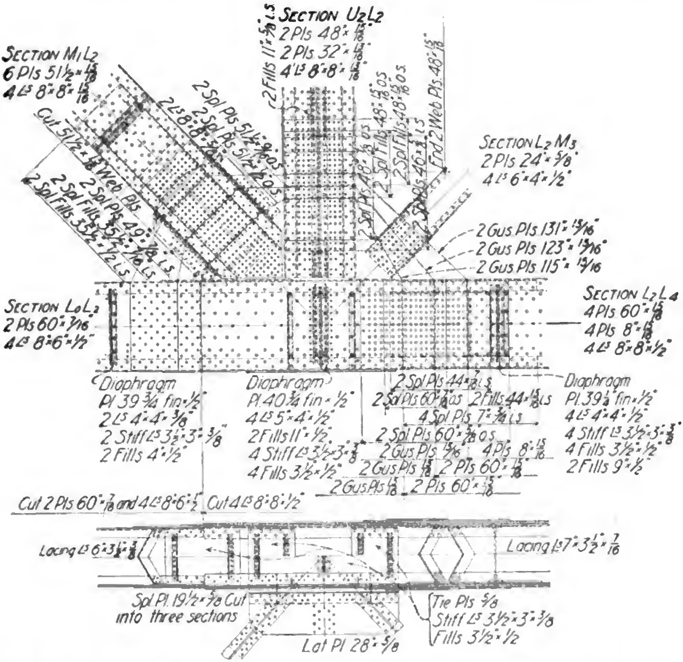


FIG. 22*rrr*. Bottom Chord Joint L_2 of the 423-foot Fixed Spans of the Fratt Bridge over the Missouri River at Kansas City, Mo.

bars in different panels, whereas, on account of painting requirements, at least one-half (½) of an inch is necessary between bars in the same panel. One-quarter (¼) of an inch, or more, should be provided between a built member and an eye-bar or between two built members, this being increased properly for projecting rivet heads.

Just as in riveted trusses, so also in pin-connected ones it is necessary to consider carefully the details at the joints in proportioning the members—and even to a greater extent. After determining the sections tentatively and arranging the packing to suit, it is necessary to select a certain size of pin for one of the joints—either U_1, L_2, or L_3, as these are usually the most highly stressed pins—and figure the thickness required for bearing and the bending moments on the pin. If the first layout fulfills all the requirements, no further calculations are necessary for this point; but it is rarely the case that a single trial will suffice, as there are too many conditions to be fulfilled to make the initial trial final. In the first place, the spacing of the members must be arranged to suit the thicknesses of metal required for bearing. Then the sections through the pin, as well as back of it, must meet the specifications in regard to built tension members, when these are used. Lastly, the pin must figure for the maximum bending moment. There is no necessity whatsoever for showing herein the method of computing the bending moment on a pin after the packing has been arranged, for that has been a standard demonstration in text-books on bridges for more than three decades—in fact, it was discussed at length in the author's first book, "The Designing of Ordinary Iron Highway Bridges," published by John Wiley and Sons in 1884. After one of the pin diameters has been determined, the others should be made of the same size, unless such an arrangement would fail to provide sufficient strength in every case or would prove to be uneconomical. In any design only a few sizes of pins should be adopted for any one truss—in fact, it should seldom be necessary to use more than three.

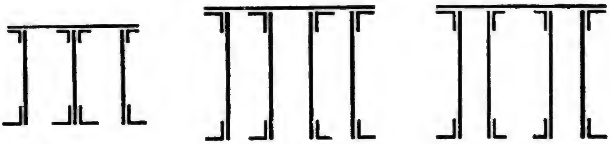


FIG. 22*sss*. FIG. 22*ttt*. FIG. 22*uuu*.

Chord Sections for Pin-connected Spans.

As far as possible the members should be arranged so as to produce small bending moments on the pins. The diagonals should be placed between the chords and the posts. The chord bars in adjacent panels should be alternated as far as possible, as the placing of two bars in the same panel adjacent to each other increases the length of the pin; but the said arrangement causes the couples from successive pairs of bars to act in the same direction, and it will sometimes be necessary to place two bars in the same panel side by side in order to reverse the effect of some of the couples. As the pin diameter is not invariably determined by bending but frequently by the bearing of the members, this matter of pin-packing may not always be vitally important, although it generally is so. At the hip the diagonal bars should be located inside of the end post and top chord, although it is sometimes necessary to place some of them outside. When this is the case, the outstanding legs of the bottom angles of these members have to be cut away, which weakens them materially at this point, necessitating either some kind of reinforcement or a slight increase in the sectional area of the member thus cut.

The pins, if it be practicable, should not have a diameter greater than the depth of the shallowest bars connected to them. Moreover, their diameter should not be less than eight-tenths (0.8) of the depth of the deepest bars that they couple, as the bearing on the pin will be about equivalent to the tension in the bar when the diameter of the former is about three-fourths (¾) of the depth of the latter, unless, perchance, the eyes be thickened. That used to be the practice of some of the manufacturing companies in the days of iron bridges, but the expedient at the present time is seldom adopted. An economical depth of plate for built tension members is about three times the diameter of the pin. Pin-plates are invariably needed for built sections. They must be sufficient to give the proper thickness for bearing on the pins, in addition to the required net section both through the pin-hole and behind it. The pin-plates should be stressed in proportion to their thicknesses, and their strength should be fully developed by the connecting rivets. When filler plates are used between angles, they should be made to serve also as pin-plates. In built tension members the sections of some of the plates through the pin-hole may not be sufficient to take the bearing stress they receive, in which case it is necessary to develop at least the difference between the two values by rivets back of the pin. In posts and other built web members, one or more of the pin-plates should extend from six (6) to twelve (12) inches inside of the end stay-plates. Moreover, when it is possible, the joints of such members should be stayed by cross diaphragms extending as near to the ends as convenient. All details must be arranged so that there shall be no interferences either in the assembling or in the field riveting.

For straight built-chords, the splices are arranged in the same manner as they are in the case of riveted trusses. They should be located to suit the erection. When the trusses are erected on falsework, the centre panel is generally placed first; and the construction then follows therefrom to the ends. It is, therefore, advisable to make this panel self-sustaining by placing the splices in the adjacent panels. The increment of chord stresses is transferred through the pins to which the web members are attached. It is necessary, of course, to splice for the section cut out by the pins. For inclined top chords breaking at the panel points, bearing should be relied on the same as in riveted trusses. Sixty per cent (60%) of the section in contact in addition to that cut out by the pins must be developed by splice-plates. At the hip all members should be made to have full bearing on the pin. In case full bearing of the top chord members on the pin at any point is relied upon, only one hinge-plate should be used. This should be riveted to the member toward the centre of the span and made long enough to be riveted to the other member after erection. In the centre panel the hinge-plates should be attached to both ends of the chord member.

Sometimes the web members are riveted together when these are of built sections. In such cases the details do not differ materially from those for riveted trusses, except for the pin connections to the chords.

The bottom laterals should be intersected on the centre line of the chords, and the posts should be extended down to engage the lateral plates. These details are described in Chapter XX.

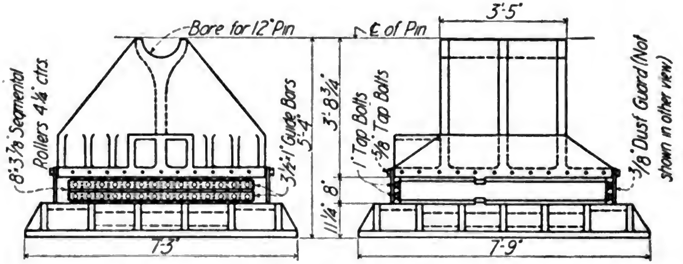

Bore for 12" Pin
℄ of Pin
3'-5"
8'-3 7/8" Segmental Rollers 4 1/4" ctrs
3 1/2"×1" Guide Bars 5'-4"
3'-8 3/4"
8"
11 1/4"
1" Tap Bolts
5/8" Tap Bolts
3/8" Dust Guard (Not shown in other view)
7'-3"
7'-9"

FIG. 22vvv. Shoe for the 423-foot Fixed Spans of the Fratt Bridge over the Missouri River at Kansas City, Mo.

The design of shoes for truss spans does not differ materially from that of those for girder spans, which are fully described in Chapter XXI, except as to the part above the rollers. This should, preferably, be a steel casting with a base plate and vertical ribs to engage the pin through the L_0 point of the bottom chord. The number of ribs required will depend on the details of the truss at this point as well as on the load to be carried. For ordinary trusses two ribs are sufficient, and these are generally placed outside of the end post. For shoes with three ribs, the division of the reactions between them can be determined from the curves in Fig. 29b; while when four ribs are employed, between each pair of which the webs of the end posts are placed, each rib can be assumed to carry one-fourth (1/4) of the total load on the shoe. The main ribs should be well braced with cross ribs; and the height of the shoe should be made as small as possible, consistent with good designing. For very heavy trusses full bearing across the pin is sometimes necessary. Such a condition arose on the 423′ riveted spans of the Fratt Bridge. The detail of this shoe is shown in Fig. 22*vvv*.

Fig. 22*fff* gives the details for the shoes of a heavy single-track-railway span.

Built shoes are sometimes employed, but they are not as satisfactory as those made of cast steel—neither are they appreciably cheaper.

A detail used on certain bridges for the protection of the trusses consists of a horizontal girder riveted to the web members above the floor. This is known as a skid girder, from the fact that derailed cars are expected to skid along it without striking the truss members. A description of the detail as used on the Beaver Bridge over the Ohio River on the Pittsburgh and Lake Erie R. R. is to be found in *Engineering Record* of May 6, 1911. While on one occasion long ago the author was called upon to design skid girders for a proposed large bridge, he has never made use of them; but he considers them favorably, and would advise their adoption for very important railroad bridges.

CHAPTER XXIII

TRESTLES, VIADUCTS, AND BRIDGE APPROACHES

THE distinction between trestles, viaducts, elevated railroads, and bridges is a difficult one to draw. The Standard Dictionary gives the following definitions:

> "Trestle. An open braced framework for supporting the horizontal stringers of a railway bridge or other structure."
>
> "Viaduct. A bridge-like structure, especially a large one of arched masonry, to carry a roadway or the like over a valley or ravine, or across another roadway."
>
> "Bridge. A structure erected across a waterway, ravine, road, or the like, serving for the passage of persons, animals, or vehicles, or as a means of support and transit, as for a water-main."

Unfortunately, the terms trestle and viaduct are used interchangeably by many engineers. It will generally be conceded, though, that a trestle is a viaduct, but that a viaduct is not necessarily a trestle. A trestle consists of a succession of towers of steel, timber, or reinforced concrete, supporting short spans, while the piers of a viaduct may be of masonry, steel, or timber, and the spans may be either long or short.

The distinction between a viaduct and a bridge is still more difficult to draw. It is certain that every viaduct is a bridge, but every bridge is not necessarily a viaduct. The general idea is that a bridge is a structure most of which is over water, while a viaduct is a structure most of which is over a dry gulch, but the existence therein of a small stream would not change the denomination. In America the term viaduct usually refers to a highway structure, while in England it applies equally to a railway bridge. Since the advent of the electric railway, the distinction between railway and highway bridges has become somewhat involved, because most of the important modern highway structures carry electric cars as well as road vehicles.

The distinction between viaducts and elevated railroads and between trestles and elevated railroads is not absolutely determined. An elevated railroad, in the common acceptance of the term, is a continuous elevated construction (generally, but not necessarily, of metal) located in a large city, usually in the streets or alleys, but sometimes on private property, and carrying trains of cars which, as a rule, are lighter than those of the steam railroads. The elevated tracks of the latter where they pass through a city and are carried either on earth fills enclosed in retaining walls or on steel construction are in reality elevated railroads, although not generally spoken of as such. If a long steel structure carrying trains is located outside of a city, it is ordinarily termed a trestle and not an elevated railroad. Finally, some structures located in cities carry both electric railway and wagon traffic. These are called viaducts and not elevated railroads. The preceding are the distinctions adopted in this treatise in regard to the four terms under discussion.

Approaches to bridges may be ordinary earth embankments finished off at the adjacent ends with either abutments or buried piers, earth fills enclosed by retaining walls, stone or concrete masonry arches, timber trestles, reinforced concrete trestles, or steel trestles. Where so-called approaches consist of short steel spans resting on masonry piers, they ought not to be considered as approaches at all, but as a portion of the bridge. However, if the structure is composed of a few long spans over a large river, flanked by a number of short spans that are supported by piers of masonry (either stone or concrete), laymen are sure to insist on calling the collection of short spans with their piers on each side of the main structure an "approach." As it is custom which establishes nomenclature, there is probably no use in trying to prevent the division of such a structure into "bridge" and "approaches," logic to the contrary notwithstanding.

The best kind of approach to a railway bridge or to a highway bridge located outside of city limits is generally an earth embankment finished off with a concrete abutment (either plain or reinforced); because such construction involves almost nothing for maintenance and repairs, excepting, of course, the pavement or macadam. For a while there will probably be a slight settlement of the earth, but in a few seasons this will cease, and the embankment will become permanent after the sides have been sodded (either naturally or artificially). Without the sodding, a certain small amount of repair work will have to be done occasionally to make good the ravages of wash.

Buried piers, which are discussed fully in Chapter XLIII, are not quite as satisfactory as abutments for finishing off the ends of embankments, but in some situations they are perfectly legitimate. Where the banks are very high they effect considerable saving in first cost; but the maintenance of embankment involved is a trifle greater, owing to the uncovered ends thereof. This item may be of importance in case that an overflow of the stream tend to erode the toe. Proper protection by riprap, however, should generally keep the damage down to an inconsiderable amount.

If the ground that would be occupied by the side slopes of an embankment be valuable or liable to become valuable in the not-too-distant future, it may be advantageous to put in retaining walls having the exterior surface vertical or nearly so. This question is one solely of economics, and must be settled for each case as it arises according to the principles given in Chapter LIII, the equivalent first cost at a number of future dates for both styles of construction being figured, and due account being taken of the variance in cost of maintenance. The differences between these figured amounts should be compared with the probable values of the land saved. This comparison will determine the economics of the case at the various dates assumed, and will enable the engineer to settle which style of construction to adopt.

Masonry arches are used for bridge approaches when it is necessary for railways or roadways to pass beneath the embankments, and especially in places where æsthetics are an important consideration. They involve expensive construction, and, therefore, are suitable only for important structures.

Timber trestles are a very common type of approach to both railway and highway bridges, their principal recommendation being their comparatively low first cost. But as timber becomes scarcer and its cost greater, the economy of wooden trestles will gradually disappear. It is often advisable to use them temporarily (for they can be built quickly as well as cheaply) with the intention of replacing them, before they decay, by earth embankments or steel construction. They can be counted upon to last from six (6) to ten (10) years, but by careful selection of timber, taking special pains in framing and erection, and the expenditure of a little more money (mainly to protect the timber against decay) three (3) or four (4) years more can be added to their life. With the present prices of steel and timber, it is nearly always cheaper to build approaches of wood and replace it with other wood as it decays. An economic study of the types of construction will show which of them is the cheaper at each of the various periods when a reconstruction of the entire timber trestle becomes necessary. If the study is assumed to extend over a long period of years, the costs of reconstruction should be figured for assumed increased prices of timber. There is a most objectionable feature inevitable in timber trestles, viz., the constant danger of being burned, to which they are subject. By creosoting or otherwise treating the timber their life may be increased severalfold, and today it is generally best to do so, as an economic study based upon a probable life of twenty-five years for the creosoted material would readily show; but, unfortunately, the creosoting appears to augment the peril from fire.

Reinforced concrete trestles are more common in highway than in railway structures, because many engineers fear that the impact on the latter is apt to injure the concrete. However, many such railway trestles are now being built, and none as yet has failed. They cost much more than timber trestles and generally more than steel ones; but when properly designed and constructed, they require very little expenditure for maintenance.

Steel trestle approaches for railway bridges are preferable to earthen ones only when the latter exceed them materially in cost, and when it would be dangerous to obstruct the waterway by building embankments.

The way to determine which of the two types it would be preferable to build, for any particular case where either would be suitable, is to make an economic study of the question covering a long series of years and taking into account the cost of maintenance.

It is sometimes necessary to spread or fan the tracks at the end of a bridge for the purpose of making a right and left turnout or to provide a storage yard for cars. In such cases, if earth embankment be too expensive or otherwise objectionable, a trestle (preferably of steel but permissibly of timber) will have to be employed. In designing it, care should be used to provide for the effect of centrifugal force, based upon an assumed velocity that ordinarily will not be exceeded. If this elevated construction has to cross tracks or roadways, the designing of it may become quite complicated.

The laying out of steel trestles is often influenced by railroads and wagon roads to be crossed. This is especially true in cities and their suburbs. In such cases the structure is nearly always low, and the spans, for economy, have to be made as short as the conditions below will permit. The first step to take in preparing such a layout is to make a map on a fairly large scale showing with extreme accuracy each track and road crossed, also the horizontal clearance lines for each. From the latter there can be seen what ground is available for the placing of columns, remembering that it is generally permissible to let the bases encroach a little on the neighboring rights of way below ground level and for a short distance above. The layout should involve as few skew spans as possible; and skews of adjacent bents should be made alike, if practicable. The question of overhead clearance is one that should be considered in making any layout, hence a profile of the crossing showing the grade of the structure and all vertical clearances is a necessity. If, at any crossing, the vertical distance between the clearance line and the grade is restricted, it will not be feasible to put in a long span there because of the great depth of girder which would be required. In such cases it is profitable to adopt girder depths that are far from economical of metal, and often as shallow as a proper consideration of the matter of deflection will permit. In many cases of restricted head room it is a good plan to adopt half-through girders; but if there are to be any track crossings or turnouts on the deck, this expedient would, of course, be impracticable. If serious difficulty be encountered because of restricted head room, a last resort is to raise the grade on the structure or to depress the grades of the tracks or roads beneath. The former method is generally preferable, because it is optional with the company constructing the structure to make the change, while permission would have to be obtained from other parties in order to adopt the latter; and such permission is usually exceedingly difficult to secure.

Where there are no tracks or roads or streams to be crossed, the layout of any trestle is a question of economics. The best possible layout in any such case is that which will make the total cost of both substructure and superstructure a minimum. The economic span lengths are a function of the height of structure, and as this varies at different places, it is evident that a great many lengths might be employed, but, for manufacturing reasons, this is not advisable; hence it is best to adopt as few span-lengths as possible. Of course, if there is an exceedingly great diversity in the heights, the span-lengths should vary, but generally the variation should occur abruptly in groups and not in small amounts from span to span, unless, perchance, this suggested arrangement should militate too seriously against an æsthetic appearance. There is an economic ratio between the lengths of tower spans and intermediate spans, which depends greatly upon the style of bracing used in the tower and somewhat upon the height.

The theory of the economics of steel trestles is discussed in Chapter LIII; and several of the diagrams pertaining to Chapter LV give data concerning the economic span-lengths for various heights and different kinds of layouts. In the excellent book of F. C. Kunz, Esq., C.E., entitled "Design of Steel Bridges," which has just been issued, there is given on p. 250 *et seq.* considerable valuable information concerning economical span-lengths for trestles and viaducts. Such data are exceedingly difficult to collect or expensive to prepare. The few diagrams of weights of metal for steel trestles contained in Chapter LV, prepared especially for this treatise, cost for computers' salaries alone six hundred dollars. The weights of metal determined by the formulæ of Mr. Kunz check quite closely with those given in the said diagrams.

For high structures the layout will necessarily consist of alternate tower spans and intermediate spans; but for low ones it is often advisable to insert solitary bents (either fixed or rocker) between the towers; and for very low structures, such as elevated railroads, there may be a succession of three or four such bents, the number depending on the span lengths and the height. With fixed ends for columns the temperature stresses run higher than with rocker ends. (See Chapter XI, Equations 39 to 42, inclusive.)

With alternating tower and intermediate spans there is nearly always one expansion point at each tower; but the author has tried the experiment of connecting rigidly two adjacent towers by riveting the intermediate girders thereto, the result being eminently satisfactory as far as rigidity is concerned. Of course, the temperature stresses run high, but as the trestle is a highway structure, and, therefore, not often subject to live loads approaching at all closely those for which it was designed, the combination of large temperature stresses and large live-load stresses is not likely to occur. If it ever does, the overstress would do no harm, as it would be well within the limit of elasticity of the steel. This feature of construction would not be so applicable to railroad trestles where great live loads are often applied. If it is ever tried on them, careful calculations of combined temperature, live-load, and dead-load stresses should be made, and the resulting intensities of actual working stress should be kept within the limits set in the specifications of Chapter LXXVIII. This expedient should not be employed for low trestles, say under seventy-five (75) feet in height; and when the tower tops are rigidly connected, the tower spans should be comparatively short so as to permit of the towers springing without causing unduly high temperature stresses in the columns.

For low trestles where solitary bents are employed, as in elevated-railroad construction, the expansion points should generally be spaced not to exceed one hundred and fifty (150) feet apart. Where a greater distance is adopted, it may prove necessary to strengthen some of the columns to resist the combination of temperature stresses with the other stresses to which they are subjected.

The hinging of pedestals avoids one-half of the trouble due to the temperature stresses caused by connecting several bents rigidly at their tops by longitudinal girders, as compared with similarly connected bents without rocker pedestals; and all the trouble can be avoided by hinging the columns at both top and bottom. Theoretically this seems to be ideally good practice, but it causes a loss of some of the rigidity which is the ultimate object of riveting several consecutive spans to the bents. Cases sometimes occur where it is necessary to put in a long stretch of low trestle without expansion, and in these the use of the suggested double-rocker bents in the neighborhood of the expansion points adopted would afford a fairly satisfactory solution of the problem.

Sliding joints are generally required at the column feet of towers; but if the distance apart be not greater than twenty (20) or twenty-five (25) feet, they may be attached rigidly to the pedestals, which would then have to spring possibly as much as three- or four-hundredths of an inch from normal position under extreme variations of temperature. Where sliding is provided, one column foot of a tower should be fixed in both directions, and one should be arranged to slide longitudinally, one transversely, and one in any horizontal direction. The column feet should be connected by substantial struts on all four faces of the tower so as to force the pedestals to slide.

In railway trestles and in important highway trestles the tower bracing should consist of struts throughout, but in unimportant highway trestles the diagonals may be adjustable rods, as this type of bracing effects a considerable economy of metal. Where stiff diagonals are employed, there should be a horizontal strut at each panel point of the bracing frame. The author recognizes that many engineers omit the horizontal struts in such bracing, and, in fact, he has done so himself in times past; but he is now convinced that the omission involves secondary stresses in the columns that are too great to ignore. It requires only a little extra metal to put in such struts; and they ought not to be omitted for any reason whatsoever, if one intends building truly first-class construction throughout the entire structure. Some engineers put in the horizontal struts and let each diagonal consist of a pair of light angles laced or stayed together, upon the assumption that they are purely tension members and cannot carry compression. As a matter of fact, one diagonal of each pair will carry a certain amount of compression which is limited only by the lateral springing of the piece, after which the other diagonal, acting in tension, takes care of the transverse shear. Unless such tension members have their connecting rivet-holes punched short of the theoretically true position so as to throw initial tension on the piece, they will be loose and vibratory, in which condition they are inferior to rods, as these can always be tightened by screwing up, while the loose, riveted diagonals cannot be adjusted without cutting out the rivets and doing a lot of expensive repair work. Where riveted bracing is a *sine qua non* and the first cost of construction has to be kept as low as possible, the angle tension-diagonals can be employed; but they are unsatisfactory for truly first-class work.

The amount of batter for tower columns varies from zero to about three (3) inches to the foot. For high, narrow trestles it is generally about one and a half (1.5) inches to the foot. The usual determining factor for the batter is the absence of tension from wind on the windward columns and their anchor-bolts when the structure is not loaded. In the author's opinion, the possibility of such tension is something in the nature of a bugbear to most bridge designers. A small amount of it can do no possible harm, provided that the pedestals have sufficient mass to resist with perfect safety the greatest possible uplift. There is no need whatsoever for increasing the sectional area of the columns because of a slight reversion of stress caused by wind pressure, because each reversion, if it ever exists at all, will come only at great intervals of time. The narrower the tower the less expensive the transverse bracing, hence it is economical to keep the width down. It is true also that the narrower the tower the greater the wind stresses on the columns; but it is not often that the wind loads influence the sectional area of these members, owing to the fact that, when the effect of wind is included, the metal can be stressed much higher than for combinations of stresses from which the effect of wind is excluded.

When trestles are on sharp curves, great batters must be used in order to provide against the overturning tendency of the combined centrifugal force and wind load. It is in such cases that the necessity arises for dividing up the transverse bracing by intermediate vertical columns so as to stiffen the long transverse struts, as will be explained presently.

It is a mistake to batter trestle columns in the planes that are parallel to the longitudinal axis of the structure, no matter how long the intermediate spans may be; because, incidentally, the lengthening of the said spans with the corresponding shortening of the spans over the towers is uneconomical, but mainly because the shopwork involved in the manufacture of a tower battered on all four faces is troublesome, expensive, and conducive to error,—to say nothing of the extra expense and the worry entailed in the drafting. The only plea for such battered towers is æsthetical; but the improvement upon the general appearance of the structure which they would effect is more than doubtful.

One of the main controlling factors in tower designing is the thrust of braked trains, for not only does it determine the sections of the members of the longitudinal bracing, but it also adds greatly to the stresses on the columns. The farther apart the columns are spaced longitudinally the smaller will be the effect of the braked-train thrust upon them, but the more expensive will be the longitudinal bracing. It is evident also that the more elaborate is the latter the shorter for economic reasons should be the length of tower spans. Clause 45 of Chapter LXXVIII specifies the traction loads which are to be used.

For highway trestles and double-track-railway trestles vertical columns can be used up to a height from pedestal to grade equal to two and a half (2.5) times the perpendicular distance between the axes of the two exterior columns composing the bent. Beyond that limit the columns should be battered. The distance between column centres for double-track-railway trestles should be so taken that the outer longitudinal girders will lie in the vertical planes containing the column axes, provided that the before-mentioned 2.5 ratio of height to width be not materially exceeded.

As previously mentioned, when trestle towers are very wide, it becomes necessary to break up the transverse sway bracing by the insertion of an intermediate column in each bent, running up only to such an elevation that the width there will be too small to require a division of bracing. Although these intermediate columns carry no live load and but little dead load, they should be provided with fair-sized pedestals so as to resist overturning properly and to give rigidity to the construction as a whole. The exact maximum width for trestle bents without intermediate columns is difficult to specify, being really a matter of individual opinion. The author would place it at about thirty-five (35) feet, which would correspond to a height of about one hundred (100) feet in a single-track-railway trestle. For extremely high trestles it is necessary again to divide the transverse vertical sway bracing by short columns resting on small pedestals. Wherever a tower is wide enough to require a division of its bents by vertical posts, there should be some light longitudinal bracing between the said vertical posts so as to stay the long horizontal transverse struts and permit of their being proportioned for reasonably small ratios of l over r.

In very high trestles the distance from centre to centre of adjacent towers is generally limited by the practicable length of the projecting boom of the traveler which does the erection. On this account it is not always feasible to adopt the span-lengths which would reduce the total weight of metal to a minimum; but it can be done, of course, by erecting the towers with gin-poles, riveting up the girders on the ground, and hoisting them into place from the tops of the completed towers. Up to the present the greatest distance between tower centres for trestles erected by traveler is about one hundred and fifty (150) feet. By the employment of the other method of erection much greater distances between tower centres can be adopted. In designing any exceedingly high trestle, careful computations of cost of finished structure should be made for each method of erection, using the most economic span-lengths for each case. To do this, one should figure not only on the cost of the metal, but also on that of the plant required to erect it. Sometimes it might be possible to procure by renting or purchase a traveler used on previous construction, but generally it will be necessary to figure on having one built. If so, its salvage value, if any, should be deducted from the cost of erection.

The greatest height of steel trestle yet erected is three hundred and twenty (320) feet from water surface to grade. Several structures of about this height have been built.

No matter how long the intermediate spans of a trestle may be, it is bad policy to make them pin-connected, because that type of truss is too vibratory for trestles. On account of their great height and slenderness everything possible should be done to make them rigid. For this reason adjustable rods and tension angle-diagonals should not be employed in their construction.

In planning long railway trestles and viaducts, provision should always be made for safety places or retreats for persons and hand-cars which may be caught upon the structure by an approaching train. The floor at and near such retreats should be planked over so that the men in moving a hand-car will have no trouble in stepping around it.

There is a common feature of steel trestle designing which the author cannot endorse, viz., cutting off the tops of columns and resting the main girders thereon, or, what is equally bad, resting longitudinal girders on top of cross-girders. First-class detailing requires that the ends of all longitudinal girders shall rivet into either the columns or the cross-girders and not rest loosely on them. Loose supports tend to set up vibration; hence they should be avoided, except, of course, at the expansion pockets.

As a rule, it is best to keep the tops of trestle pedestals within two or three feet of the surface of the ground. If they project far therefrom and are built with the usual batter, they will look too flimsy for first-class work. If a high projection be unavoidable, the batter should generally be made greater so as to increase the solidity of the construction, and thus check vibration from either the live load or the wind pressure—or from both combined.

In proportioning masonry or concrete pedestals, the weight should not

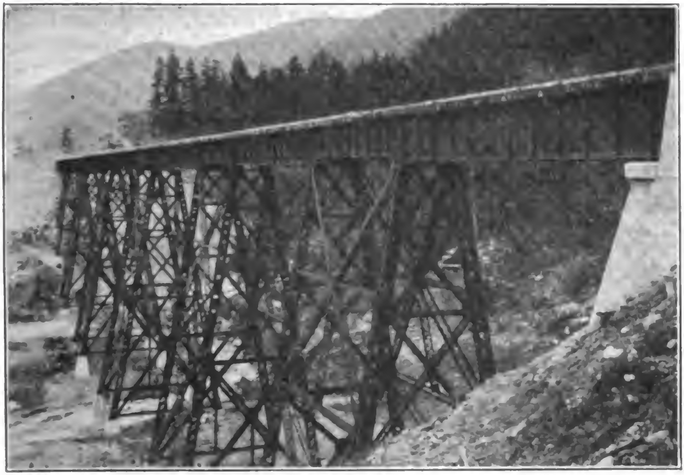


Fig. 23*a*. Single-track-railway Trestle over Stoyoma Creek near North Bend, B. C., on the Line of the Canadian Northern Pacific.

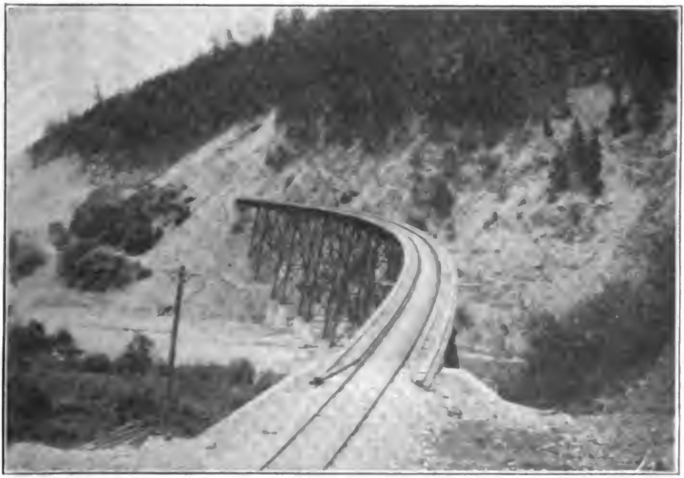


Fig. 23*b*. Single-track-railway Trestle over Anderson Creek near North Bend, B. C., on the Line of the Canadian Northern Pacific.

be made less than twice the greatest possible net uplift that can come upon the column foot under any condition of loading, due account being taken of the buoyant effort of the water in case of possible submergence of the pedestal.

The anchor-bolts should be so attached to the masonry as to develop their full strength before pulling out. Generally it will be the wind pressure on the unloaded trestle that will produce the greatest net uplift, but it may come from a combination of wind and centrifugal force, or of

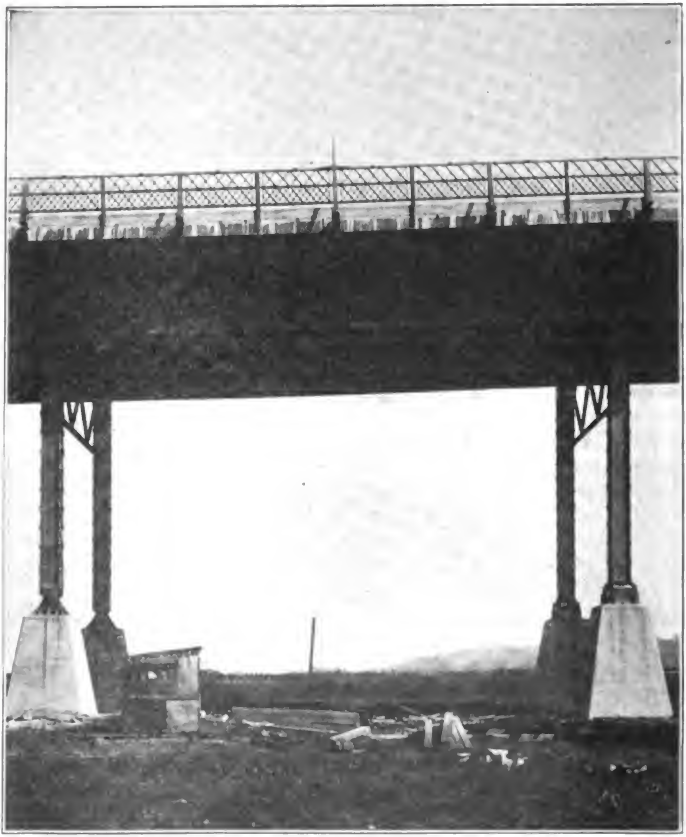


FIG. 23c. Solitary-column Bents in the North Approach of the Fratt Bridge over the Missouri River at Kansas City, Mo.

wind and thrust of braked train. A combination of the three loads is practically impossible, hence it need not be considered.

When the old Kinzua Viaduct was renewed in 1900, a new type of tower was employed, which, to say the least, was bizarre. Its special feature was the omission of the X-bracing in the bents and compensating therefor by curved brackets in the corners formed by the intersection of the horizontal cross-struts with the columns. Whether the object of the variation from the current practice was economy of metal or desire to produce something new is not apparent. If any metal were saved, its value was much more than offset by the expensive shopwork involved in the curved brackets; and certainly there must have been a loss of rigidity, because the brackets are not as stiff as crossed diagonal struts. Again, the computations of wind stresses are much more intricate than those for bents with X-bracing, and the secondary stresses must be higher. To the author there does not appear to be any good reason for the omission of the X-bracing. If it were advisable to omit it from the bents, why was it retained in the longitudinal bracing? Would not the corner brackets have served just as well there as where they were used? The fact that this type of construction has not been repeated during the past decade is fairly good (although not conclusive) evidence that it did not receive the general approval of the engineering profession.

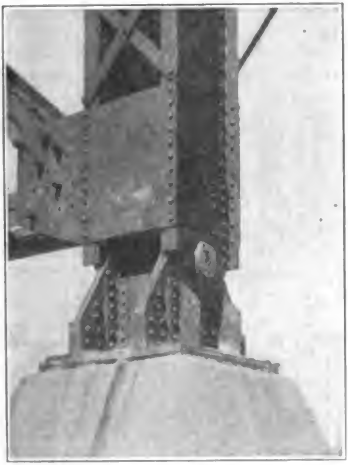


Fig. 23*d*. Built Rocker-shoe for a Solitary Bent in the South Approach of the Fratt Bridge over the Missouri River at Kansas City, Mo.

In Skinner's "Details of Bridge Construction" will be found a compilation of a mass of valuable information about bridge approaches, viaducts, and trestles, illustrating many of the principal constructions both in America and abroad, to which the reader is referred for data of a historical nature concerning such structures.

The general character of the steel railway trestles designed by the author and representing his latest practice is shown in Figs. 23*a* and 23*b*, representing two structures on the line of the Canadian Northern Pacific Railway in British Columbia, where that road parallels closely the Fraser River. One trestle is located on tangent, the other on curve. The character of all the main members and even of some of the details can be recognized from these photographs. It had been the intention to supplement this chapter with a number of detail drawings covering both the sections of main members and the various connections; but the issuing of Mr. Kunz's book has obviated the necessity for some of this, because in his Chapter XII he presents several pages of detail illustrations; and the reader is referred to these. However, the author disagrees with Mr. Kunz on three rather important points, viz.:

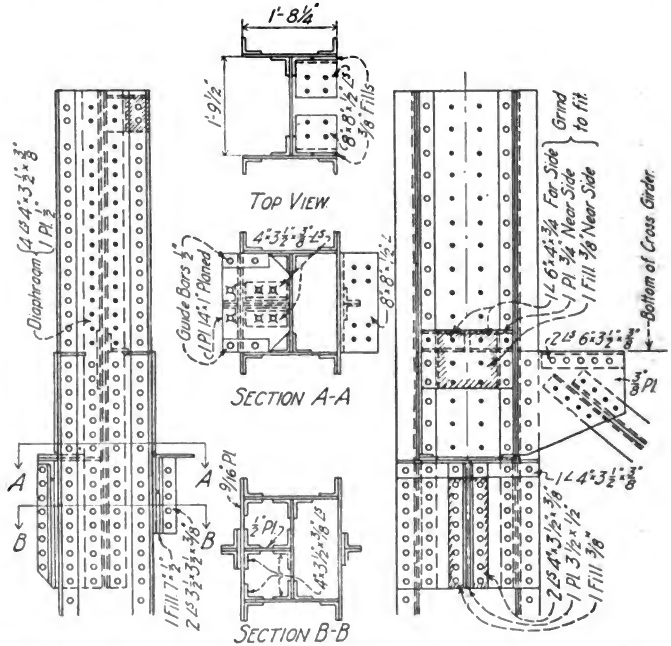


FIG. 23*e*. Column Expansion Pocket for a Girder in the South Approach of the O.-W. R. R. & N. Company's Bridge over the Willamette River at Portland, Ore.

First. He does not approve of single-angle diagonals in towers and bents.

Second. He objects to the omission of the horizontal struts at the panel-points of the vertical sway bracing, because of the high secondary stresses that their omission causes.

Third. He rivets the longitudinal girders and the end cross-girders into the tops of the columns, which are bent into vertical position for that purpose, as shown in Fig. 23*a*. The weakening effect of the bend is fully compensated by the brackets connecting rigidly the portion of the column immediately below the bend to the end cross-frame.

Fig. 23*c* shows two solitary bents in the North Approach to the Fratt Bridge over the Missouri River at Kansas City, Mo. This approach consists of towers about three hundred and forty (340) feet apart on centres with intermediate single bents between them. Full-depth transverse bracing could not be employed, and consequently the columns and the anchorages had to be designed for the transverse wind bending. The bent at the left of Fig. 23*c* has a rocker base, while that on the right has a fixed base. A cast-steel shoe was used for the former, but the fixed shoe was built of plates and angles.

Fig. 23*d* shows a built shoe for the rocker base of one of the columns of the South Approach to the same bridge. As seen from the illustration, the view was taken during construction.

In Fig. 23*e* is given the detail of the expansion pocket for a girder in one of the approaches to the O.-W. R. R. & N. Co.'s bridge over the Willamette River at Portland, Ore.

CHAPTER XXIV

ELEVATED RAILROADS

NOTWITHSTANDING the popular opinion that elevated railroads are an offense to the eye, an annoyance to the ear, and a general nuisance in every sense, it must be admitted that they are a necessary evil, for residents of cities simply must have some means of rapid transit between their homes and their places of business. The only alternative method of securing it is by subways, and these are objectionable for several reasons: They are far more expensive in first cost, the ventilation in them invariably is bad, and people, as a rule, object to going under ground in advance of the time when it becomes absolutely necessary. It is easy enough to point out the objectionable features of elevated railroads, but it is exceedingly difficult to remedy them. To beautify them, or even to render them not displeasing to one's artistic taste, is about as feasible as the proverbial difficulty of making a silk purse out of a sow's ear. As for deadening the noise from passing trains—that has been tried a number of times, but not very successfully. If the deck were planked over with creosoted timber, or, preferably, were covered with a slab of reinforced concrete, and upon this were laid ballast to carry the ties, much of the noise would be suppressed—as much probably as it is possible to prevent; but such a floor would certainly be expensive, and, in the case of the planked floor, the percolation of dirty water through the ballast and plank on to the passers-by beneath would have to be prevented by the expenditure of a considerable amount of money. A ballasted floor was used on some three or four miles of the Market Street Elevated R. R. of Philadelphia.

To attain truly rapid transit on an elevated railroad, there should be at least four tracks, and the determination of the alignment should be left to the engineers and not to the purchasers of the right-of-way. The line should be as nearly absolutely straight as practicable, and the curves at the turns should be made as easy as the conditions existing there will permit. Nearly all such turns in the track are of necessity very sharp, consequently passing around them keeps down the speed; and if there are many turns, it is evident that after leaving one of them the velocity cannot be increased very much before another one is reached; hence ideally rapid transit under such conditions would be an impossibility. To secure the best results in transporting passengers quickly, the two inner tracks should be used for express trains only and the two outer ones for local trains, the stations for the express traffic being spaced about a mile and a half from each other and those for the local traffic about a quarter of a mile apart. The platforms at the local stations should be outside of all the four tracks, and those at the express stations between the outer tracks and those adjoining them, the two inner tracks being left straight and the outer ones being curved around the said platforms. Passengers should be taken in at the ends of the cars and discharged from the middle. Incoming and outgoing passengers should be kept separate at the stations. It is best not to let the latter pass through the station building at all, but make them descend to the street by exit stairways through turnstiles.

Elevators and escalators at the stations where the traffic is most dense not only tend to hasten travel but also to reduce the effort of the passenger in reaching his train. Unfortunately, though, they are expensive in both first cost and operation, especially the elevators, which have to be operated by attendants. As the escalators move continuously, no attendant is needed; but the power required for operation is large.

If it be impracticable for lack of funds or other reason to build a four-track elevated railroad, rapid transit may still be secured by constructing a double-track elevated with a double-track surface line directly beneath, running the express trains above with stations a mile or two apart, and transferring the passengers to the surface line. This method is nearly always feasible, and it is ideally economic; consequently it seems strange that it has not yet been adopted.

In figuring on any elevated railroad project where the line runs through private property, especially if the structure is to have four tracks, one should consider carefully the advisability of building thin retaining walls of reinforced concrete, tying their tops together at short intervals by adjustable rods encased in a covering of concrete, and filling the boxed spaces with earth. If the steel market be high, this type of construction might cost less than a steel elevated road; and if it cost but little more, it would prove cheaper in the end, because of the much smaller cost for its maintenance and because, as far as is known today, it is imperishable.

When the tracks occupy city streets, the stations are generally elevated, although they might be located at street level in adjacent buildings with stairways leading to the platforms. But when the tracks occupy private property, the proper place to locate each station is on the said property beneath the structure directly adjacent to a street-crossing.

About the time the author completed his work on the Northwestern Elevated Railway and the Union Loop Elevated Railway of Chicago, he wrote for the American Society of Civil Engineers an exhaustive paper on Elevated Railroads. It was discussed very thoroughly by a large number of the members; and both the paper and the discussions were published in the *Transactions* of the Society for 1897. The paper called attention to the numerous defects in the then existing elevated railroads of the United States and pointed out their remedies, referring to the author's designs for the Chicago elevated roads as an example of what he considered proper detailing. Later, as consulting or advisory engineer to the Boston Elevated Railroad Company he outlined the method of detailing that structure. Several other large elevated railroads have been constructed since then; but, as far as can be learned from the technical press and by systematic correspondence on the part of the author with the engineers of these new roads, no material improvements in the designing and construction of elevated railroads have been effected since the Chicago and Boston lines were built.

There was given in Chapter VIII of *De Pontibus* a compendium of the before-mentioned paper on elevated railroads, and as the author has nothing further to add to the treatment of the subject, it is reproduced herein as follows, a few slight modifications being made in the wording so as to bring the treatment of the subject up to date.

LIVE LOADS

The proper live load to assume in designing an elevated railroad is the greatest that can ever come upon it, and is determined by ascertaining the weights of engines and empty cars that are adopted at the outset, then computing how many passengers can be crowded into the latter, and assuming that the average weight per passenger is one hundred and forty pounds. The live loads for elevated railroads, unlike those for surface railroads, do not increase from time to time, but remain constant. In fact, electric operation has rather decreased them, for the weight on the axles of a motor-car generally produces smaller bending moments than that on the axles of a steam locomotive.

After the distribution of the live load on the various axles of the entire train has been determined, it is well to prepare a diagram of equivalent uniform loads and one of total end shears similar to those given for the standard live loads of Chapter VI, in order to facilitate the computing of stresses and bending moments.

FLOOR

The style of floor in general use on elevated railroads consists of timber ties with four lines of timber guard-rails, closed floors of buckled plate or reinforced concrete carrying timber ties in ballast being employed at crossings of important streets and boulevards so as to prevent dirt and moisture from falling upon people passing beneath. Such a closed floor has been advocated for the entire line, and certainly it would be an improvement upon the open one; but the increased expense involved is likely to interfere seriously with its adoption for future elevated railroads. The ballast over the buckled plate in tight floors is necessary to prevent noise from passing trains, which, unless some effective sound-deadener be adopted, would be simply deafening.

ECONOMIC SPAN-LENGTHS

With the ordinary live loads, for structures located on private property the economic span-length is about forty feet, while for structures located in the street it varies from forty-seven to fifty-three feet according to the transverse distance between vertical axes of columns, the greater the distance the greater the economic span-length. With heavier live loads the economic span-lengths would be shorter.

FOUR-COLUMN VERSUS TWO-COLUMN STRUCTURES

In four-track structures located on private property there is but little, if any, difference in the cost whether four columns or two columns per bent be employed; but preference is given to the former on account of its greater rigidity.

BRACED TOWERS VERSUS SOLITARY COLUMNS

In structures on private property there is quite a gain in both rigidity and economy by adopting braced towers spaced about one hundred and fifty feet centres.

RAILS

The author prefers to adopt for elevated railroads steel rails five inches high weighing not less than eighty pounds to the yard, so as to provide for the excessive wear caused by the constantly passing trains.

TREATED VERSUS UNTREATED TIMBER

Extended investigations have proved that it pays well to preserve the track timber, and that, up to the present time, by far the best preservative process is creosoting.

PEDESTAL-CAPS

The most satisfactory and economical pedestal-caps are of concrete covered with at least six inches of first-class granitoid. These have all the advantages of cut-stone blocks, and are generally cheaper. The latter, however, can be used if there be anything to be gained thereby, provided that the quality of the stone be first class in every particular.

ANCHORAGES

It is best to anchor columns to pedestals by means of anchor-bolts, either two or four per column—according to whether there is bending on the latter in one or in two directions—extending well down into the concrete and well up outside of the column, to which they connect by means of long enclosing plates and heavy washers. The boxed spaces at the column feet should always be filled with concrete to prevent the collection of dirt and moisture.

PLATE GIRDERS VERSUS OPEN-WEBBED, RIVETED GIRDERS

As far as economy goes, there is no material difference between plate-girder work and open-webbed, riveted work; but the former is more satisfactory in most particulars, the only advantage claimed for the latter being that it is more sightly. On this account it is preferable for structures occupying the streets, while plate-girder work is more advantageous for those located on private property.

CRIMPING OF WEB-STIFFENING ANGLES

Investigation has shown that it is economical to crimp the intermediate stiffening angles and best to use fillers beneath the end stiffeners.

SECTIONS FOR COLUMNS

The best section for columns located in the street is one composed of two channels with their flanges turned inward and an I-beam riveted between them, the flanges of the former being held in place by interior stay-plates spaced about three feet centres. The main object in turning the flanges inward is to enable the column better to resist impact from heavily loaded vehicles. The most satisfactory section for columns located on private property consists of four Z-bars and a web plate.

EXPANSION JOINT

The author's ideal expansion pocket, which was described very fully by both text and drawings in his paper on Elevated Railroads, is reproduced later in this chapter.

PROPER DISTANCE BETWEEN EXPANSION POINTS

With columns fixed at both top and bottom, the proper distance between expansion points is about one hundred and fifty feet.

SUPERELEVATION ON CURVES

Superelevation of the outer rail can be obtained by varying the heights of the stringers, by putting a wooden shim on the outer stringer, by using bevelled ties, or by spiking a shim to each tie. The last two methods are generally preferable, but the second is not considered to be good practice, while the first one would give unnecessary trouble in the shops. It will generally suffice to employ only three bevels for ties, viz., one, two, and three inches in five feet. Such bevels will not, it is true, afford the theoretical superelevation required for the maximum speed on sharp curves; but it must be remembered that it is difficult to maintain high speed there, hence the compromise between theory and practice.

FAULTS IN EXISTING ELEVATED RAILROADS

In concluding his before-mentioned paper, the author made a list of the principal faulty details in existing elevated railroads, thereby provoking much animated discussion; and, as the subject is one of great importance to the designer of future similar structures, that portion of the paper which includes this list will be reproduced here nearly verbatim.

I. INSUFFICIENCY OF RIVETS FOR CONNECTING DIAGONALS TO CHORDS OF OPEN-WEBBED, RIVETED GIRDERS

This defect is more noticeable in old structures than in later ones, especially as the tendency nowadays is very properly to substitute plate-girder for open-webbed construction. In many of the older elevated roads there is no connecting plate between the diagonal and the chord, but one leg of each of the angles in the diagonal is riveted directly to the vertical legs of the chord angles. This detail involves the use of either two or four rivets to the connection, which is evidently very bad designing, as there should be more rivets employed, even if the diagonal stresses do not call for more on purely theoretical considerations. Where the theoretical number of rivets is very small, additional rivets should be used for two reasons, viz., first, one or more of the rivets are liable to be loose, and, second, there is nearly always a torsional moment on each group of rivets, owing to eccentric connection.

II. FAILURE TO INTERSECT DIAGONALS AND CHORDS OF OPEN-WEBBED GIRDERS ON GRAVITY LINES

It is very seldom indeed that the designer even attempts to intersect at a single point all of the gravity lines of members assembling at an apex. The failure to do so involves large secondary stresses, especially in the heavier members. By using connecting plates, it is always practicable to obtain a proper intersection; and it is better to do this than to try to compensate for the eccentricity by the use of extra metal for the main members.

III. FAILURE TO CONNECT WEB ANGLES TO CHORDS BY BOTH LEGS

Some standard bridge specifications stipulate that in case only one leg of an angle be connected, that leg only shall be counted as acting, although this stipulation is generally ignored by the designer working under such specifications. It is seldom, indeed, that both legs are connected. In order to settle the question of the necessity for this requirement, the author made, in connection with his Northwestern Elevated work, a series of tests to destruction of full-sized members of open-webbed girders, attached in the testing machine as nearly as practicable in the same way as they would be connected in the structure. It was intended to settle by these tests the following points: first, the effect of connecting by one leg only; second, the effect of eccentric connection; and, third, the ultimate strength of star struts with fixed ends, each of these struts being formed of two angles. The principal deduction to be made from the tests is that an equal-legged angle riveted by one leg only will develop about 75 per cent of the strength of the entire net section, while a 6″ × 3½″ angle riveted through the longer leg will develop about 90 per cent. It is, therefore, more economical for short diagonals to use unequal-legged angles connected by the longer leg than to employ supplementary angles to try to develop the full strength of the piece. In fact, the experiments made up to date indicate that these supplementary angles will not essentially strengthen the diagonal. These tests showed also that the star struts mentioned do not develop as much strength as one would suppose, and, therefore, that they cannot be recommended for the construction of steel structures. Later experiments on single angles and two angles riveted together back to back, connected by riveting to an exterior plate, developed only fifty per cent, or even less, of the theoretical strength by the column formula, when tested in compression. The specifications of Chapter LXXVIII cover this feature of proportioning such struts.

IV. FAILURE TO PROPORTION TOP CHORDS OF OPEN-WEBBED, LONGITUDINAL GIRDERS TO RESIST BENDING FROM WHEEL-LOADS IN ADDITION TO THEIR DIRECT COMPRESSIVE STRESSES

This neglect is common enough in the older structures, and the fault is a serious one, although the stiffness of the track rails and the flexibility of the ties tend to distribute the load and thus to reduce the bending.

V. INSUFFICIENT BRACING ON CURVES

Too often in the older structures the curved portions of the line are no better braced than are the straight portions. A substantial system of lateral bracing on curves extending over the entire width of the structure and carried well into the tops of the columns adds greatly to the rigidity of the construction and, consequently, to the life of the metalwork.

VI. INSUFFICIENT BRACING BETWEEN ADJACENT LONGITUDINAL GIRDERS

The function of the bracing between longitudinal girders is an important one, for it is the first part of the metalwork to resist the sway of trains. Not only should the top flanges of adjacent girders be connected by rigid lateral bracing, but the bottom flanges should be stayed by occasional cross-bracing frames, one of the latter being invariably used at each expansion end of each track. The bracing frames between the girders of adjacent tracks, if any be used there, should have no diagonals; for these would transfer some of the live load from one pair of girders to the other and would not only overstress some of the metal when one track only is loaded but also would induce torsional stresses on the unloaded pair of girders.

VII. PIN-CONNECTED, PONY-TRUSS SPANS AND PLATE GIRDERS WITH UNSTIFFENED TOP FLANGES

These defective constructions are noticeable in some of the older lines, but, fortunately, not often in the newer. What the ultimate resistance of the pony-truss structure is no man can tell without testing it to destruction; but, in the opinion of most engineers, it is much less than it is assumed to be by those designing pony-truss bridges.

VIII. EXCESS OF EXPANSION JOINTS

Too many expansion joints in an elevated railroad are nearly as bad as too few. In the former case the metal is overstressed by the vibration induced by the lack of rigidity, while in the latter case it is overstressed by extreme variations of temperature. There are elevated roads in existence with expansion joints at every other bent, and there is at least one with them at every bent. For long spans expansion should generally be provided at every third bent, and for short spans at every fourth bent.

IX. RESTING LONGITUDINAL GIRDERS ON TOP OF CROSS-GIRDERS WITHOUT RIVETING THEM EFFECTIVELY THERETO

This is by no means an uncommon detail, especially in the older structures. It is conducive to vibration, and its only advantages are ease of erection and a cheapening of the work by avoiding field-riveting.

X. CROSS-GIRDERS SUBJECTED TO HORIZONTAL BENDING BY THRUST OF TRAINS

The resistance that can be offered by a cross-girder to horizontal bending is very small; nevertheless, cross-girders are rarely protected from the bending effects of thrust of trains. What saves them from failure is the fact that the continuity of the track tends to distribute the thrust over a number of bents. However, it is not legitimate to depend on this, for, especially on sharp curves, the tendency is to carry the thrust into the ground as directly as possible. In the author's opinion, the only proper way to provide for this thrust is to assume that 20 per cent of the greatest live load between two adjacent expansion points will act as a horizontal thrust upon the columns between these two expansion points; and to proportion all parts of the metal-work to resist this thrust properly. By running a strut from the top of each post diagonally to the longitudinal girder at a panel-point of its sway-bracing, the horizontal thrust is carried directly to the post, and a horizontal bending moment on the cross-girder is thus prevented. Such construction should invariably be used where the conditions require it.

XI. CUTTING OFF COLUMNS BELOW THE BOTTOMS OF CROSS-GIRDERS AND RESTING THE LATTER THEREON

This style of construction, which until the early nineties was almost universal, is extremely faulty in that there is no rigidity in the connection and that the column is thus made more or less free-ended at the top. It has been said that no harm is done to the column by making it free-ended, as it can then spring better when the thrust is applied. Unfortunately, this reasoning is fallacious, because the few unlucky rivets which connect the bottom of the cross-girder to the top of the column tend to produce a fixed end, and are, in consequence, racked excessively by the thrust of the train. In all cases the column should extend to the top of the cross-girder, and should be riveted to it in the most effective manner practicable.

XII. PALTRY BRACKETS CONNECTING CROSS-GIRDERS TO COLUMNS

Brackets are often seen composed of a couple of little angles attached at their ends by two or three rivets. Such brackets are merely an aggravation, and are sure to work loose sooner or later. Although it is impracticable to compute the stresses in this detail, good judgment will dictate the use of solid-webbed brackets riveted rigidly to both cross-girder and column so as to stiffen the latter and check the transverse vibration from passing trains.

XIII. PROPORTIONING COLUMNS FOR DIRECT LIVE AND DEAD LOADS AND IGNORING THE EFFECTS OF BENDING CAUSED BY THRUST OF TRAINS AND LATERAL VIBRATION

The practical effects of this fault can be seen to best advantage by standing on one of the high platforms of one of the elevated railroads of New York City. The vibration, by no means small, from an approaching train can be felt when it is yet at a great distance. Some may claim that this vibration is not injurious; but they are certainly wrong, for what does it matter, so far as the stress in the column is concerned, whether the deflection be caused by vibration or by a statically applied transverse load, so long as the amount of the deflection is the same in both cases? It takes metal, and considerable of it, to make columns strong enough to resist bending properly; and a sufficient amount should be used to attain this end.

XIV. OMISSION OF DIAPHRAGM WEBS IN COLUMNS SUBJECTED TO BENDING

If the diaphragm web be omitted in such a column, reliance must be placed on the lacing to carry the horizontal thrust from top to bottom. But even if the lacing figure is strong enough to carry it, which is unusual, it is wrong to assume it so, for the reason that one loose rivet connecting the lacing-bars will prevent the whole system from acting, as will also a lacing-bar that is bent out of line. Decidedly, every column that acts also as a beam should have solid webs at right angles to each other.

XV. INEFFECTIVE ANCHORAGES

For the sake of both rigidity and strength, every column ought to be anchored so firmly to the pedestal that failure by overturning or rupture would not occur in the neighborhood of the foot, if the bent were tested to destruction. The flimsiness of the ordinary column-foot connection is beyond description.

XVI. COLUMN-FEET SURROUNDED BY AND FILLED WITH DIRT AND MOISTURE

The condition of the average column-foot is simply deplorable. This is caused by failing to raise it high enough above the street to prevent dirt from piling around it, and by omitting to fill its boxed spaces with concrete. When rusting at a column-foot is once well started, it is almost impossible to stop it from eating up the metal rapidly.

XVII. INSUFFICIENT BASES FOR PEDESTALS

False ideas of economy on the part of the projectors and indifference on the part of some unscrupulous contractors occasionally cause the use of pedestal bases altogether too small for the loads that come upon them, especially where the bearing capacity of the soil is low. The result is sunken pedestals and cracked metalwork. In figuring this pressure on the base of the pedestals, it is not sufficient to recognize only the direct live and dead leads, but it is necessary also to compute the additional unequal intensities of loading caused by both longitudinal and transverse thrusts. Impact also should be considered.

Concerning the question of the extent to which the faults just outlined exist in the older elevated railroads of this country, the author would refer the reader to the résumé of discussions on his paper.

About the most important object to attain in constructing an elevated railroad is to have a perfectly smooth and durable track; and no trouble nor expense should be spared to secure it. For this reason the top flanges of the longitudinal girders, if the limiting heights of grade and clearance line permit, should be several inches higher than those of the cross-girders, the ties should all be planed to exact dimensions, tie-plates should be used over all ties, and the system of bolting of flooring to structure should be the most effective possible. The longitudinal girders should not be made continuous, or even semi-continuous, over the cross-girders; and when blocking up is necessary, short buckled plates should be placed over the latter so as to provide a continuous surface for the ties. Hook bolts should be used for attaching the timber to the metalwork through each alternate tie, the other ties being bolted to the inner guard-rails.

The ties should be spaced with openings not greater than six inches, their section for a five-foot stringer spacing being 6″ × 8″ laid on flat; but where cross-covers are employed, the depth should be properly increased to withstand the bending moment due to the greatest load from the wheels.

The least allowable overhead clearance for most cities is fourteen feet; but there are sometimes special crossings requiring a greater height. The width of right-of-way beyond the centre line of the outer track should not be less than seven feet.

The proper depths of longitudinal girders are to be determined very carefully. For the sake of appearance it is generally not well to use more than one depth, but such an arrangement cannot always obtain. The general depth should, if possible, be the economic one for the average span-length. For plate-girder spans it is about one-tenth of the length, while for open-webbed, riveted spans, it is much greater—so much greater, in fact, that for deck spans the economic depth cannot be adopted, because of the raising of the grade whichwould be caused thereby.

Before the designing of the metal work for an elevated railroad is started there are certain important matters which should be fully determined, viz., the dimensions and weights of rolling stock, sizes and number of trains, method of traction and the proper track to suit the same, the locations of all stations and their leading dimensions, the storage capacity for the terminals, the capacity of the repair-shops, and the method of operating the road. Unless all these questions be settled conclusively at the outset and before the designing is begun, trouble is sure to ensue because of changes that will have to be made from time to time during the course of both design and construction.

The faulty details criticized in the preceding extract from *De Pontibus* have been corrected in the latest elevated railroads, although the struts from longitudinal girders to distant columns have sometimes been omitted—to the evident detriment of the structure as far as rigidity is concerned.

Mr. William Barclay Parsons, in designing the West Side Viaduct extension of the New York subway, evolved a new type of column. It consists of four bulb angles, rolled especially for the subway construction, with a single web-plate, which web-plate is omitted in the upper part of the column. The web-plate of the cross-girder extends downward from the bottom flanges thereof, and is of the same thickness as the web-plate of the column. This projecting web is dropped down between the bulb angles of the columns and is riveted thereto. The section of the column is economic, and the connection is a good one, although somewhat more difficult, perhaps, to make in the field than the ordinary attachment of cross-girder to column. The rivets were arranged for driving by power machines in the field, and their number was smaller than those for the usual type of connection.

With possibly the exception of a few modifications of minor importance in the deck, as far as the author can learn, this is the only detail of elevated railroad designing which is in the nature of an improvement over those adopted in the building of the before-mentioned elevated railroads in Chicago and Boston.

There is a detail in the Boston Elevated Railroad that, when adopted, was assumed to be an improvement on the Chicago construction; but the author does not agree with the assumption. It consists in inserting between the top of the concrete pedestal and the steel column-foot a heavy cast block to distribute the pressure, the anchor-bolts passing idly through it and taking hold of the sides of the column. There is no need of interposing anything between the concrete and the steel foot, if the latter be properly spread by adequate stiffening; and it is always difficult to ensure sufficiently perfect contact between the squared end of a strut and a casting. It takes more metal and more workmanship thereon to put in the casting than to finish the foot as suggested, and the actual result obtained is inferior because of the loose block between the column and its bearing. Moreover, it adds unnecessarily to the length of the anchor-bolts. In short, there does not appear to be any good reason whatsoever for adopting such a detail, unless the required bearing area at the column base be unusually large.

The weights of metal used per lineal foot of single track, according to the author's practice in 1893, were approximately as follows:

With pedestals directly under tracks and with longitudinal bracing, 400 lbs.

With pedestals directly under tracks and without longitudinal bracing, 450 lbs.

With columns straddling a double-track street railway and no longitudinal bracing, 500 lbs.

With columns on curb lines and straddling a 40-ft. street and without longitudinal bracing, 600 lbs.

The live load corresponding to the preceding weights varied from 2,000 lbs. per lineal foot of track for a length of 30 ft. to 1,160 lbs. for a length of 190 ft.

Curiously enough, the specifications employed in designing the Chicago elevated railroads more than two decades ago, although impact was not then considered, give almost identical proportions for members as those that would be obtained by using the specifications of Chapter LXXVIII and the impact for electric railway bridges given in Fig. 7*d*.

On page 269 of Kunz's "Design of Steel Bridges" the weight of metalwork in elevated railroads carrying light (passenger) traffic is stated to be about 500 lbs. per lineal foot per track, and that for standard railroads about 2,000 lbs. for double-track and 3,000 lbs. for four-track structures. These figures are based on a wooden tie flooring; and it is stated that, if steel flooring and ballast are used, about fifty per cent should be added thereto.

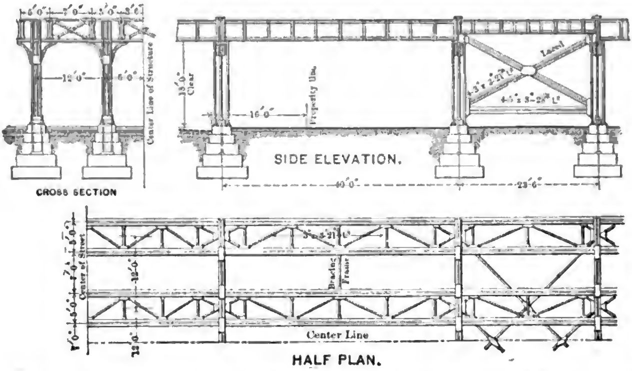


FIG. 24*a*. Plan, Elevation, and Cross-section for a Four-column Elevated Railroad.

The author is inclined to doubt the correctness of this last statement; for if it will apply to a light passenger elevated railway it will not to a standard steam railway, as the weight of the buckled plate or trough floor is about the same in the two cases, as is also the amount of ballast. If a ballasted floor is to be employed, it would be better and cheaper to support it on a slab of reinforced concrete.

The characteristic detailing for elevated railroads as given in the author's before-mentioned paper on that subject is herewith reproduced. Had any material improvement been effected in their designing during the last two decades, the old drawings would have been discarded and new ones would have been prepared.

Fig. 24*a* shows a cross-section, side elevation, and plan for a four-column structure erected on private right-of-way. It is the best and most economic type that can be used. Fig. 24*b* shows an enlarged half elevation of a tower. A possible improvement in the detailing might have been effected by making the upper corner bracket connect to the shallow longitudinal girder; but the detailer evidently considered that the end of the deep longitudinal girder stiffened the connection effectively for carrying the horizontal component of the diagonal stress into the shallow girder.

Fig. 24*c* gives a cross-section of the floor adopted on the two Chicago elevated railroads.

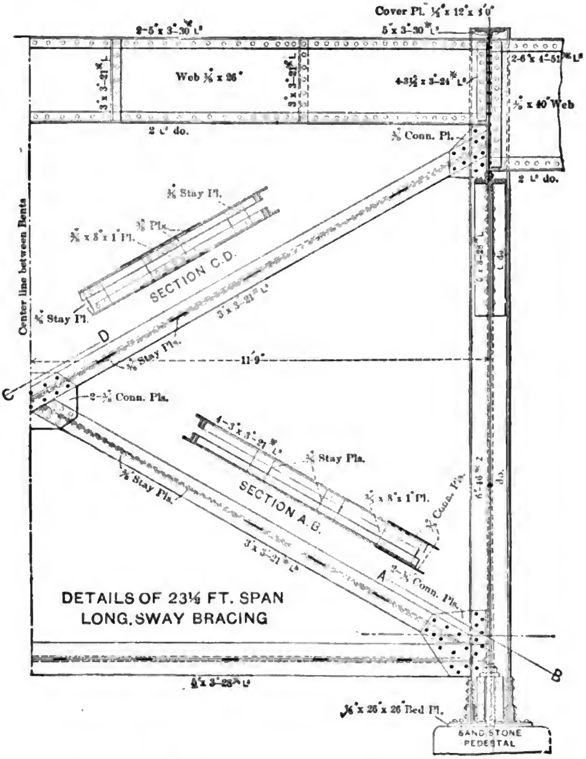


Fig. 24*b*. Half Elevation of Tower for Elevated Railroads.

Fig. 24*d* gives a detail for a column-foot and a pedestal with the connection between them. This type of detail, which makes the column and pedestal act as one continuous whole with no tendency under any possible circumstances to part at the joint, was first employed by the author in 1891 when designing the columns and pedestals of a train-shed at Sioux City, Iowa.

Fig. 24*e* gives a cross-section and a horizontal section of the elevated structure just below the deck at a fixed end; and Fig. 24*f* illustrates the same views at an expansion end.

Figs. 24*g* and 24*h* show on a larger scale the expansion pocket at the sliding ends of the longitudinal girders. The method of figuring the rivets for the eccentric connection of the said pocket to the column is explained fully in Chapter XVI.

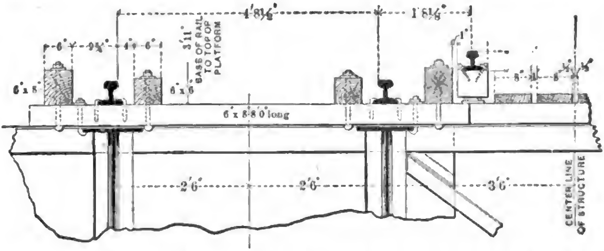


FIG. 24*c*. Cross-section of Floor for the Northwestern Elevated Railway and Union Loop Elevated Railway in Chicago.

For further information as to elevated railroads the reader is referred to the before-mentioned paper in the *Transactions* of the American Society of Civil Engineers for 1897, and to Chapter XIII of Kunz's "Design of Steel Bridges."

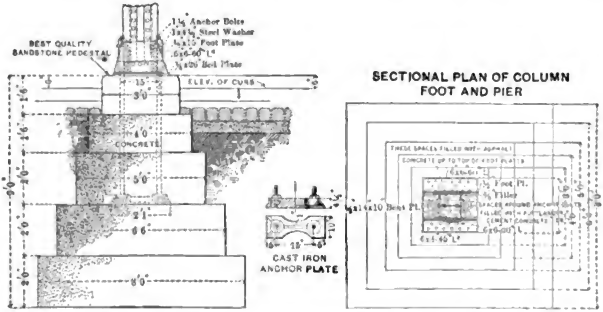


FIG. 24*d*. Detail of Column Foot and Pedestal for Elevated Railroads.

Since the preceding was written there has appeared in *Engineering News* of May 20, 1915, an excellent article by Maurice E. Griest, Esq., C.E., Assistant Designing Engineer of the Public Service Commission of New York City, entitled "Design of Steel Elevated Railways, New York Rapid Transit System," and describing the lines now being built, most of the designing work on which was done under his immediate supervision.

This is the latest and best type of construction for elevated railways, although it does not contain many improvements upon the Chicago elevated railroad work herein described. There is one detail that is certainly an improvement, viz., the insertion of a half-round sliding bearing in the expansion pocket instead of the thick plate used formerly. It permits the girder to deflect under load without any tendency to shift the point

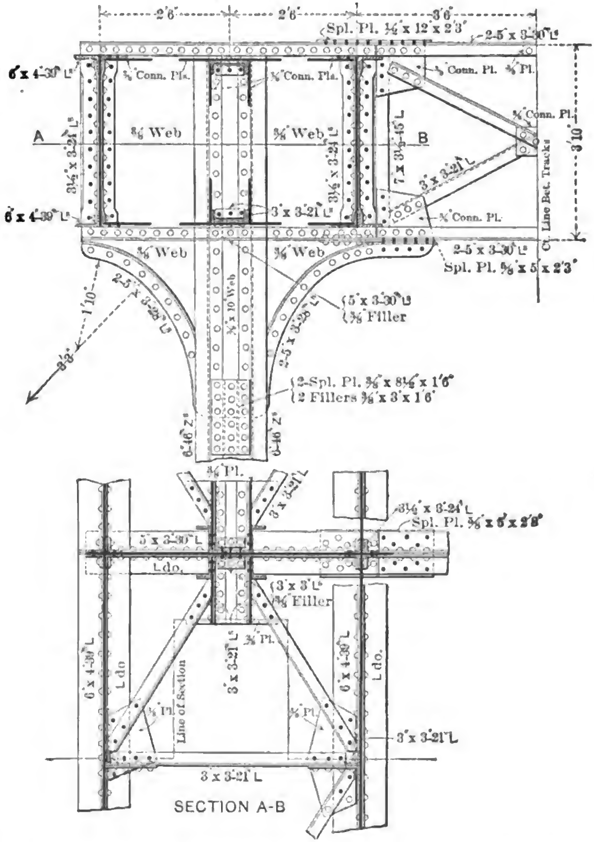


FIG. 24e. Details at Upper Part of Column with Fixed Girders for Elevated Railroads.

of application of the end reaction. The connection of longitudinal girders to tops of columns located on curb by means of deep, stiff struts, as inaugurated by the author for the Chicago structures, has very properly been adopted.

There are several points of general interest in Mr. Griest's paper well worthy of noting. One of them is that the ratio of total cost of line for open floor and solid floor is as one is to one and six-tenths. This is certainly a great difference, and shows conclusively that it is better to use the open-floor construction wherever the conditions will permit. Mr. Griest shows, as did the author, that there is considerable advantage in placing the columns in the same longitudinal vertical plane as the outer longitudinal girders and inserting corner brackets beneath them so as to obtain additional rigidity.

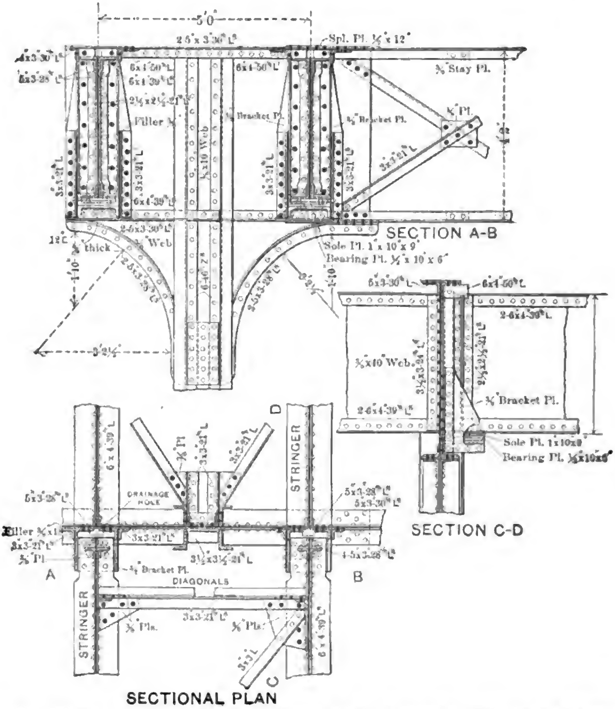


FIG. 24*f*. Details at Upper Part of Column with Expansion Girders for Elevated Railroads.

Mr. Griest's structures all carry three tracks, while those of the author have either two or four; hence it is difficult to give a proper comparison of weights of metal. Again, the recorded weight of metal, 2,250 lbs. per lineal foot of structure, includes the steel in the stations. A fair allowance for this would be 200 lbs. per lineal foot, leaving 2,050 lbs. per

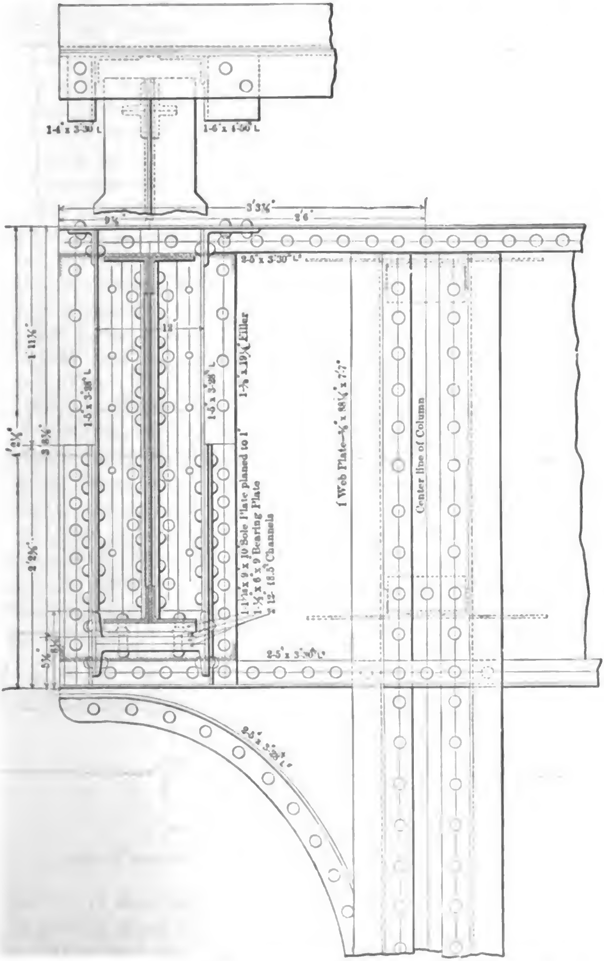


FIG. 24*g*. Details of Expansion Pocket for Elevated Railroads.

lineal foot for the structure proper, or 683 lbs. for each track. As nearly as his records show, a three-track structure in Chicago with columns on curbs would have required about 1,500 pounds of metal per lineal foot

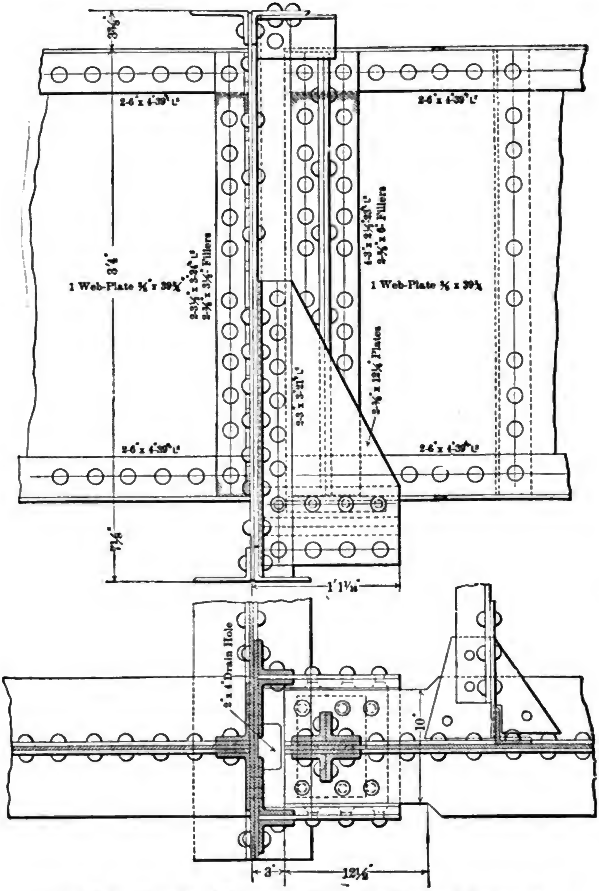


FIG. 24*h*. Details of Expansion Pocket for Elevated Railroads.

of span, or 500 lbs. per track. The ratio of live loads in the two cases for a 50-ft. span is about 1.6, and that of total loads about 1.5. This increase would add not more than twenty-five per cent to the author's weight of metal, making it 750 lbs. per lineal foot for like loading as against 683 lbs. This is a fairly close check for such necessarily rough figuring, and shows that Mr. Griest's designing was economically done.

Another variation in detailing is the increasing of the relative height of the longitudinal girders so as to let their top flanges pass half-way over the cross-girders. This practice was established on the South Side Elevated Railway of Chicago some years ago by C. V. Weston, Esq., C.E. (later the president of that company), who was associated with the author in the fieldwork of his Chicago elevated railways. It is somewhat of an improvement upon the old method of dropping the tops of the longitudinal girders below the upper flanges of the cross-girders; but there never was any serious objection to the latter arrangement.

Mr. Griest gives a diagram showing that for structures located in the street the economic span-length is fifty feet, but that for lengths from forty-five feet to fifty-five feet there is not much difference in the cost. This agrees exactly with the author's investigation made some twenty years ago; for in his paper on "Elevated Railroads" he stated that "The investigation showed that for plate-girder construction through private property the economic span-length was about forty feet, while for similar construction in the street it varies from forty-seven to fifty feet, or even three or four feet more in case of cross-girders spanning wide streets from curb to curb."

Mr. Griest adopted for his columns the same section that the author used on the Union Loop Elevated, viz., two fifteen-inch channels with their flanges turned inward, separated by a fifteen-inch I-beam; but finding that the latter were sometimes slightly warped and that the columns built with the defective sections were in wind, he decided to replace them by built I-beams. As these weigh more and cost more per pound than the rolled sections, the change is not in the line of economy. If the final rolling of the section be done with proper care, it will be true to shape. No trouble of this kind was experienced on the Chicago elevated railroads before mentioned.

CHAPTER XXV

CANTILEVER BRIDGES

CANTILEVER bridges are a type of structure eminently suitable for certain conditions, but they should never be adopted unless those conditions exist, because they are inferior in rigidity to simple truss bridges and usually require more metal for their construction. Generally speaking, they are appropriate only for long spans, although there is one case of short-span structure where this type may be used to advantage, viz., in crossing railroad tracks with deck plate-girder spans when the distance from clearance to grade near the middle must be made a minimum and a smaller clear headway is permissible near the ends.

The conditions which generally call for cantilever construction are deep gorges to be crossed by single spans, and the impracticability of using falsework because of danger from washout. For some years there was a slight tendency in American cities to build cantilever bridges because of their novelty and, possibly, because the city fathers were inveigled into adopting them by smooth-tongued bridge agents.

If there be assumed a river crossing of very great length, in which the bed rock is approximately horizontal and where the conditions affecting erection are not unusually dangerous, there is no possible layout for a cantilever bridge which will be as inexpensive as a structure consisting of simple truss spans of equal length, provided that the said length be the most economic one possible and not greater than six hundred feet, excepting only the highly improbable case of the simple spans necessitating a width between central planes of trusses materially greater than that called for by the traffic requirements. That this fact is not generally known is proved by the occasional building of a cantilever bridge in a place where the conditions do not call for one. For instance, there was no good reason whatsoever for making the great Poughkeepsie Bridge a cantilever structure, because by using the same number of piers and making all the spans simple and alike the cost of the substructure would not have been at all increased, but probably diminished; while the weight of metal in the superstructure and towers would have been lessened a little. It is true that there might have been a little increase in cost of falsework, but as the materials thereof could have been used several times, it would not have been large; while, partially to offset it, there is the extra cost of the adjusting apparatus for the cantilever structure and the greater cost of erection due to delays in making the central connections.

Moreover, alternate simple spans could have been erected without falsework by the expedient first adopted by the author for several Japanese bridges, which expedient will be described subsequently in this chapter.

The author has never been able to discover any good reason for making the Thebes Bridge over the Mississippi River of the cantilever type. It was claimed at first to be more economic of materials, but the author has good reason to know that such a claim is wrong. Certainly a simple span layout, such as was submitted in the competition, would have been far more sightly and no more expensive.

If a deep, narrow gorge with rocky sides has to be bridged, the cantilever construction will often prove economical for two reasons: first, the main piers, being small, are comparatively inexpensive; and, second, the cost of falsework will be greatly reduced, only a small amount thereof being used for erecting the anchor-arms.

Again, if a stream is to be bridged where it is impossible to put in falsework, or where there would be danger of its being washed out in case it could be put in, the cantilever will prove an economic design, although for certain conditions the cantilever-arch design described in Chapter XXVI may be still more economical and possibly more rigid. This last feature, however, will depend somewhat upon the character of the arch adopted.

That a cantilever bridge is less rigid and deflects more vertically than a simple span bridge no one who has examined both types of structure under load and who has computed the vertical deflections can well deny; nevertheless this comparative lack of rigidity is no great detriment or weakness, and should not be allowed to militate against the building of a properly designed cantilever bridge where the conditions call for such a structure. Compared with a suspension bridge, a cantilever bridge is rigidity itself. But, again, this is no reason for condemning *in toto* suspension bridges, which have their legitimate place in engineering construction, viz., where either an extremely long span is necessary or where a cheap highway bridge over a wide river is required.

There is one kind of steel structure in which the cantilever is more economical of metal than the simple span, viz., roofs supported on steel columns, as in train-sheds and workshops. The reason for this economy is the shortening of the spans and the ignoring of the effects of reversion of stress when proportioning members. The latter is legitimate within certain limits because of the infrequency or improbability of such reversion.

There is a case of possible economy in the cantilever type of construction in crossings which are suitable for the simple span layout, to which the reader's attention should be called. It is entirely dependent upon the specifications for the minimum perpendicular distances between central planes of simple and cantilever trusses and upon whether the width required for clear roadway has to be increased because of the great span-lengths adopted. The old-established practice has been to make the minimum distance for simple spans one-twentieth (1/20) of the length and that for cantilever-spans one twenty-fifth (1/25) or one twenty-seventh (1/27) thereof; but at present there is a tendency to increase the latter even to or beyond the limit for simple span bridges. In the Quebec Bridge which failed the ratio was about one over twenty-five (25), while in the new design it is about one over twenty (20). In the original study which the author made when the bridge was first proposed, the span-length was sixteen hundred (1,600) feet and the width over piers seventy-five (75) feet, making the ratio about one over twenty-one (21). In the Forth Bridge the ratio is one over fourteen (14). There is nothing to show that there is any truth in the newspaper statement that the narrowness of the ill-fated Quebec Bridge had anything to do with its downfall. The author believes, though, that while a limiting ratio of one over twenty-five (25) is suitable for a span of seven hundred (700) feet or under, it should be increased gradually for longer spans so that for eighteen hundred (1,800) feet it would be one over twenty (20). He does not consider that any such abnormal width as that employed for the Forth Bridge is warranted for any conditions whatever, provided, of course, that suitable foundations could be obtained for the pedestal piers were a smaller width adopted.

But to return to the question of that possible economy: let us assume that a highway bridge for which the clear roadway must be twenty (20) feet is to be built over a very wide river where the economics indicate that for simple spans a span-length of six hundred (600) feet is necessary, which length would require a width of thirty (30) feet, while only twenty-three (23) feet are needed for clearance. If we consider that a width of one twenty-seventh (1/27) of the span is permissible, we could adopt a cantilever layout having main spans as long as six hundred and ninety (690) feet, for which the least width is twenty-five and a half (25.5) feet. This layout would permit of shortening each pier four and a half feet; and it is conceivable that the consequent saving in cost of substructure would be larger than the net increased cost of superstructure.

The following is a statement of the various stresses for which the several spans of any cantilever bridge should be figured:

STRESSES IN SUSPENDED SPAN

First: Dead-load Stresses.
Second: Live-load Stresses.
Third: Impact-load Stresses.
Fourth: Direct Wind-load Stresses.
Fifth: Transferred-load Stresses.
Sixth: Erection Stresses from Dead Load.
Seventh: Erection Stresses from Wind Load.

STRESSES IN CANTILEVER-ARMS

First: Stresses due to Dead Load on Suspended Span.
Second: Stresses due to Live Load on Suspended Span.
Third: Stresses due to Impact Load on Suspended Span.

Fourth: Stresses due to Wind Load on Suspended Span.
Fifth: Stresses due to Transferred Load on Suspended Span.
Sixth: Stresses due to Erection of Suspended Span and caused by the Dead Load.
Seventh: Stresses due to Erection of Suspended Span and caused by the Wind Load.
Eighth: Stresses due to Dead Load on Cantilever-arm.
Ninth: Stresses due to Live Load on Cantilever-arm.
Tenth: Stresses due to Impact Load on Cantilever-arm.
Eleventh: Stresses due to Wind Load on Cantilever-arm.
Twelfth: Stresses due to Transferred Load on Cantilever-arm.
(This load affects only the main inclined posts over piers.)

Stresses in Anchor-Arms

First: Stresses due to Dead Load on Suspended Span.
Second: Stresses due to Live Load on Suspended Span.
Third: Stresses due to Impact Load on Suspended Span.
Fourth: Stresses due to Wind Load on Suspended Span.
Fifth: Stresses due to Transferred Load on Suspended Span.
Sixth: Stresses due to Erection of Suspended Span and caused by the Dead Load.
Seventh: Stresses due to Erection of Suspended Span and caused by the Wind Load.
Eighth: Stresses due to Dead Load on Cantilever-arm.
Ninth: Stresses due to Live Load on Cantilever-arm.
Tenth: Stresses due to Impact Load on Cantilever-arm.
Eleventh: Stresses due to Wind Load on Cantilever-arm.
Twelfth: Stresses due to Dead Load on Anchor-arm.
Thirteenth: Stresses due to Live Load on Anchor-arm.
Fourteenth: Stresses due to Impact Load on Anchor-arm.
Fifteenth: Stresses due to Wind Load on Anchor-arm.
Sixteenth: Stresses due to Transferred Load on Anchor-arm.

Stresses in Main Central Spans

Chord Stresses

First: Stresses due to Dead Load from both Suspended Spans and Adjacent Cantilever-arms.
Second: Stresses due to Live Load covering both Suspended Spans and Adjacent Cantilever-arms.
Third: Stresses due to Impact for the latter case.
Fourth: Stresses due to Wind Load on both Suspended Spans and both Adjacent Cantilever-arms.
Fifth: Stresses due to Transferred Load on both Suspended Spans.
Sixth: Stresses due to Dead Load on Main Central Span.
Seventh: Stresses due to Live Load on Main Central Span.
Eighth: Stresses due to Impact Load on Main Central Span.
Ninth: Stresses due to Wind Load on Main Central Span.
Tenth: Stresses due to Transferred Load on Main Central Span.

Web Stresses

First: Stresses due to Dead Load on both Suspended Spans and both Cantilever-arms.
(These will be zero for a symmetrical structure.)

Second: Stresses due to Live Load on one Cantilever-arm and the adjoining Suspended Span.
(This loading produces a constant shear from end to end of Main Central Span.)
Third: Stresses due to Impact from last load.
Fourth: Stresses due to Transferred Load on one Suspended Span.
(This loading produces a constant shear from end to end of Main Central Span.)
Fifth: Stresses due to Dead Load on Main Central Span.
Sixth: Stresses due to Live Load on Main Central Span.
Seventh: Stresses due to Impact from last load.

For certain conditions some of these stresses will not need to be considered, but in other cases they will; consequently it is necessary to insert them in the lists. For instance, in the cantilever- and anchor-arms the sixth and seventh items will generally be found to have no influence on the sections of members, but in some cases they will, as in long-span highway bridges with light live loads.

In calculating erection stresses, the weight of the traveler must not be forgotten, as its influence on such stresses is by no means inconsiderable.

The combination of the various stresses requires both judgment and care, for some loads may or may not act together, and some produce tension while others produce compression in the same member. Again, distinction must be made between groups of stresses with wind-stresses and those without, so as to use the different intensities of working stresses given in the specifications of Chapter LXXVIII. It would be too tedious to give here the various combinations of stresses for each member of each span; but it will suffice to say that the computer will have to find for each main member in the entire bridge the greatest tension when wind-stresses are included, the greatest tension when they are excluded, the greatest compression when they are included, and the greatest compression when they are excluded, taking care not to group together any stresses that cannot exist simultaneously.

The determination of the proper live load per lineal foot for any member of a cantilever is one requiring a little care, the rule being that for the piece considered the length of span to be used in applying the equivalent uniform live-load diagram is the total length of structure which must be covered by the moving load in order to obtain the greatest stress in the said piece, excepting only the suspended span and the main central span, for which the live loads actually imposed are to be treated exactly like those of simple spans. Of course, the impact is to be figured for the length of structure that must be covered by the live load to produce the greatest stress in the piece under consideration.

Some young engineers have an idea that the finding of stresses in cantilever bridges is a complicated matter. On the contrary, it is very simple, as every stress can be determined by the ordinary principles of statics and very readily by the use of graphics. Although the work is simple, it is somewhat long and tedious, as is evident from the preceding lists of stresses. The computer is advised, when finding the stresses, not to try to group the loadings any more than they are grouped in the said lists; for, if he does, he will probably have to separate them while making his combinations.

In respect to combinations of stresses during erection, there will be no necessity for increasing the sections proportioned for other combinations, provided they are as large as those required by the said erection-stress combinations with the intensities given in the specifications (Chapter LXXVIII) for combinations that include wind-stresses, viz., intensities thirty per cent higher than those for combinations without wind-stresses.

It is no easy matter to give an artistic effect to a cantilever bridge; nevertheless it is generally within the realm of possibility to do so, although it must be confessed that most of the existing structures of this type are uncompromisingly ugly. If a convex upward curve can be placed in the top chord of the suspended span, so as to reverse at the ends into a concave upward curve on the cantilever arm, a graceful effect will be obtained; but the design generally will not be economical for erection on account of the large erection-stresses near the point of suspension. The author once made a design on these lines for a proposed 1,500 ft. span highway bridge to cross the Mississippi River at St. Louis; and, as the suspended span would have been erected on falsework, there was no lack of economy involved. The layout with all the main members drawn true to scale has a very pleasing effect, as can be seen in Fig. 25*a*.

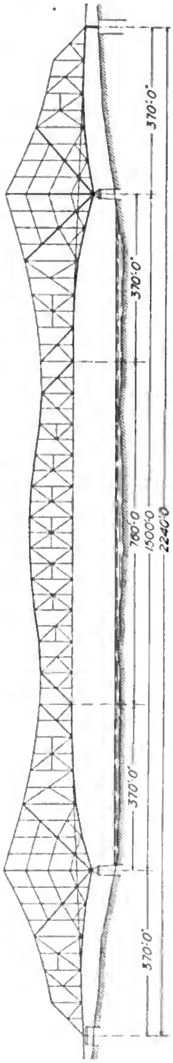


FIG. 25*a*. Proposed Highway Cantilever Bridge of 1,500 Feet Span across the Mississippi River at St. Louis.

In long spans like the one last mentioned it often becomes necessary to widen each cantilever-arm and each anchor-arm uniformly from outer end to main pier, so as to obtain the requisite rigidity for resisting wind-pressure and so as to keep the wind-stresses in bottom chords within reasonable limits. It seldom pays, however, to build the trusses of these arms in planes inclined to the vertical, principally because of the complicated shopwork involved.

For both æsthetic and constructive reasons it is better to adopt a single vertical post over each main pier than to use either two posts close together or to divide the load between two piers, as was done in the Forth Bridge. (This criticism refers only to the North and South Queensferry piers; for the two-hundred-and-sixty-foot span at the Inchgarvie pier is an anchor-span, as can be seen in Fig. 25*k*.) The single post involves greater simplicity than the double one, and is therefore preferable. When the latter is adopted, the two columns should either be entirely independent of each other or should be connected by bracing that is incapable of transferring any vertical load from one column to the other. When the double pier is used, there should be no diagonal web members in the span between them, for such members involve continuity, the absence of which is supposed to be a fundamental characteristic of cantilever construction. It is the custom in America not to employ such web diagonals in cantilever bridges; but in Europe, where continuous spans are not objected to, they are sometimes adopted.

The author once had occasion to design a number of large bridges for a proposed branch-line of the Nippon Railway of Japan. The line, which was to be about one hundred miles long, was to follow the course of the Akano River, a mountain torrent that rises from twenty to twenty-five feet in two or three hours, and attains in places a depth of water exceeding one hundred feet with a total rise of sixty feet. Of course, falsework could be employed for these bridges only to a very limited extent; hence it was necessary to resort to the use of the cantilever. Three of the eight structures were designed as ordinary cantilevers, two as simple-truss bridges, and three as cantilevers during erection and simple spans afterward. The last style of bridge is very economical of both metal and money, and will bear further investigation and extension, so as to be made applicable to crossings where the ordinary cantilever bridge would otherwise be adopted. Its mode of construction is as follows:

At each side of the river there is erected on falsework a simple span having its chords and certain of its web members (for short spans all of them) stiffened for erection stresses. Then over each pier is built a toggle consisting of horizontal upper-chord eye-bars and adjustable verticals, by means of which one-half of the central span is cantilevered over the stream to meet the other half, after which the toggles are removed. This method of erection can be understood by reference to the diagram in Fig. 25*b*.

One of the three cases mentioned had rather peculiar conditions, which necessitated the adoption of another expedient. About mid-stream there is a narrow rocky island that reaches to about the elevation of extreme high water. Near the edges of this island, as shown in Fig. 25*c*, were located two small piers, each of which was to support one end of a long span. Between the end shoes was run a temporary strut, and from each pedestal was sprung a temporary post to support the temporary top-chord eye-bars that ran from hip to hip. The rectangular panel was braced with temporary adjustable diagonals, and the top chord was hinged at the middle and connected to the pedestals by other temporary adjustable rods. These two sets of adjustable rods permit of the raising or lowering of one span at a time. By means of this device more than one-half of each span could be cantilevered out to meet the remainder thereof, which was to be erected on falsework. It was intended to erect the cantilevered portion of all three bridges with their ends higher than they would be in their final position, so that no raising, but only lowering, of weight of the arms by the toggles would be necessary.

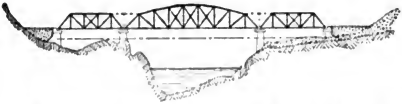


Fig. 25*b*. Layout of a Simple-truss Bridge Erected as a Cantilever.

In one of the three true cantilever bridges for the proposed Japanese railroad an expedient was adopted which may be worthy of description. One approach to the structure, as shown in Fig. 25*d*, was through a tunnel ending in the face of a vertical wall of rock. It was at first intended to use this rock in lieu of one anchor-arm of an ordinary cantilever by letting the main posts lie close to its vertical face and tying the top chords well back into its mass; but a study of the contours of the rock showed that it dipped off to one side of the line in such a way as to render such an anchorage of uncertain strength, hence it was decided to increase the lengths of the suspended span and the far cantilever-arm sufficiently to cut out the near cantilever-arm, and thus let the end of the suspended span roll on two small pedestals at the mouth of the tunnel. Five-eighths of this span were to be erected by toggles fastened into the rock, and the remaining three-eighths were to be cantilevered out also by toggles from the end of the far cantilever-arm. This method required more metal than did

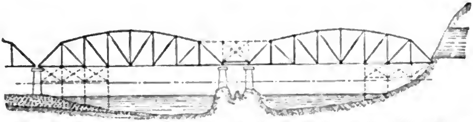


Fig. 25*c*. Layout of a Simple-truss Bridge Partially Erected by the Cantilever Method.

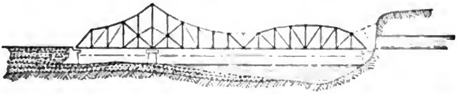


Fig. 25*d*. Layout of a Cantilever Bridge of Special Design.

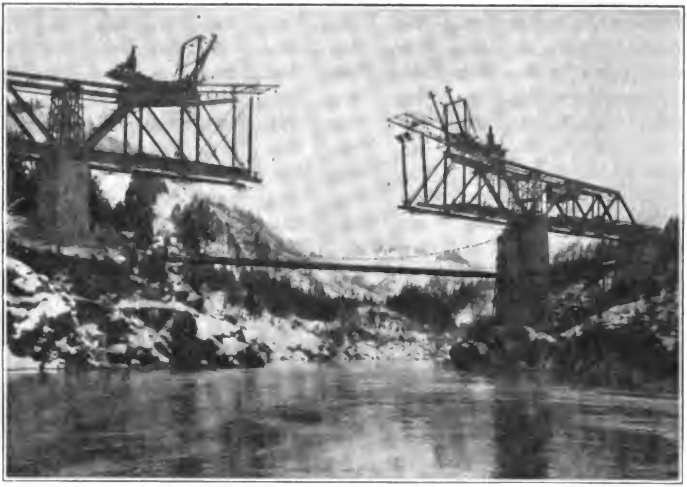


FIG. 25*e*. Simple-truss Bridge on the Nippon Railway in Japan, Erected by the Cantilever Method.

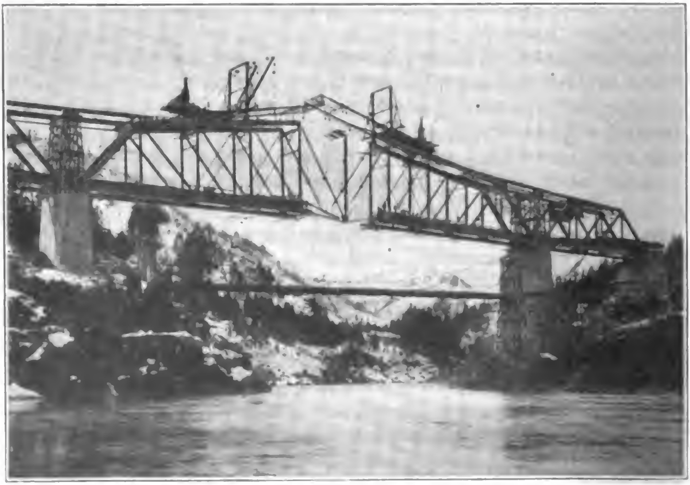


FIG. 25*f*. Simple-truss Bridge on the Nippon Railway in Japan, Erected by the Cantilever Method.

the one first contemplated; nevertheless it was the cheapest, everything considered, that could be adopted. The rock-anchorage was amply strong for the dead-load pulls on it during erection, although, as before stated, it was not sufficiently reliable for resisting the effects of live loads.

Owing to financial considerations the building of this branch line of the Nippon Railway was not undertaken for many years after the author's plans were finished; and when the project was resurrected new surveys were made, and the crossings of the river were more or less changed in consequence, thus rendering useless all the old bridge plans. However, the Japanese engineers who prepared the new designs adhered to the author's method of semi-cantilevering in several cases. One of their bridges is illustrated in Figs. 25*e* and 25*f*, which show quite effectively the *modus operandi* of the scheme of "semi-cantilevering." Although the author evolved it some two decades ago, it was not until within the last four years that he had occasion to employ it in actual construction. Bridges Nos. 2, 4, and 5 (the first mentioned across the Fraser River and the other two across the Thompson) on the line of the Canadian Northern Pacific Railway in British Columbia were erected by this means. Fig. 25*g* shows bridge No. 2 in course of construction, and Fig. 25*h* gives a view of the finished structure for bridge No. 5. The larger part of the span in the latter was cantilevered out by tying back to the high rock above the tunnel, as previously described for one of the Japanese bridges.

The best method of attaching the suspended span of an ordinary cantilever bridge is by hangers from inclined end posts on the cantilever-arms. For such suspenders narrow eye-bars should be used; and it is generally better to hinge them at the middle. This is because they are subjected to transverse bending, due to longitudinal expansion and contraction of the suspended span from both changes of temperature and the application and removal of the live load. Narrow bars can spring slightly without being overstressed, and a rotation of the eyes on the pins will thus be prevented. Such a rotation would eventually enlarge the eyes and cut notches into the pins, necessitating for some future time expensive repairs.

The author has long desired to have an opportunity to try what would be the effect on the general appearance of a cantilever bridge of leaving out the idle members of the top chord, and thus accentuating the actuality of the cantilever construction, as shown in Fig. 25*i*. This is in accordance with Precept No. 7 enunciated in Chapter LII, which treats of æsthetics. To the uninitiated such a departure from established practice might give an air of instability to the structure; but the trained eye would be pleased by the striking truthfulness of the innovation.

In cantilever-arms it is better and more economical to use inclined posts as well as vertical ones over the piers, so that the various loads will be carried more directly to the masonry. To insure the travel of the wind-stresses down the transverse bracing between these inclined

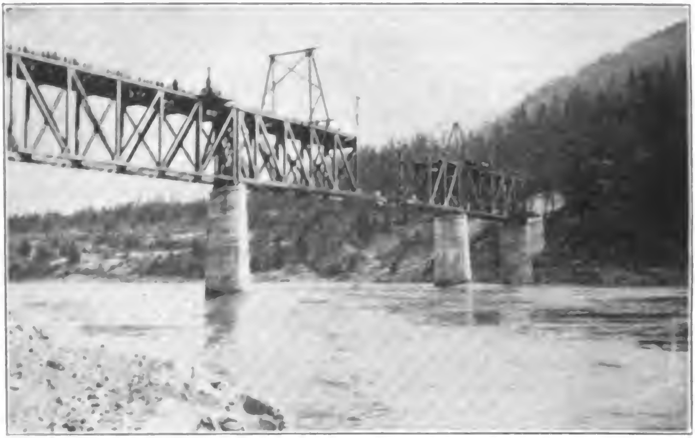


Fig. 25*g*. Simple-truss Bridge over the Fraser River on the Line of the Canadian Northern Pacific Railway in British Columbia, Erected by the Cantilever Method.

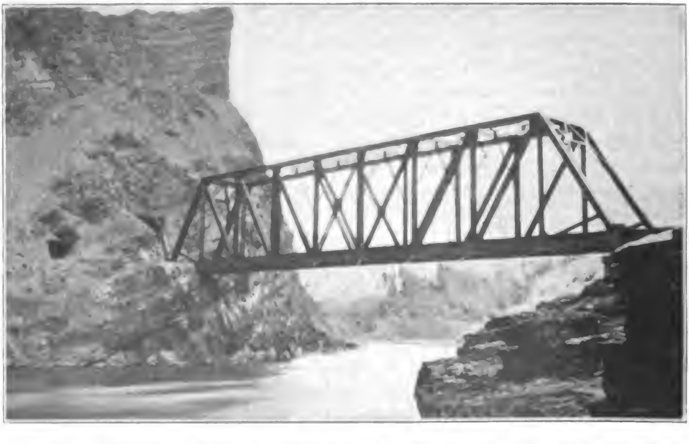


Fig. 25*h*. Simple-truss Bridge over the Thompson River on the Line of the Canadian Northern Pacific Railway in British Columbia, Erected by the Cantilever Method.

posts, instead of up to the apex of the top chord and down the bracing between the vertical posts, the author leaves out one pair of diagonals of the upper lateral system between the said apex and the tops of the inclined posts. The same expedient is used also for the anchor-arms and between the hips of the suspended span and the cantilever-arms. All bracing between opposite vertical posts and between opposite inclined posts should be made very rigid. Great care is necessary in designing the pedestals over the main piers so as to carry the loads from the three heavy posts to the masonry without overstressing any of the metal in the pedestal, and so as to distribute the total pressure uniformly over the masonry bearing.

The anchorage details require special attention, and no rules can be given to govern their designing, for the reason that the conditions vary for all crossings. The following hints, though, may be of use to the designer.

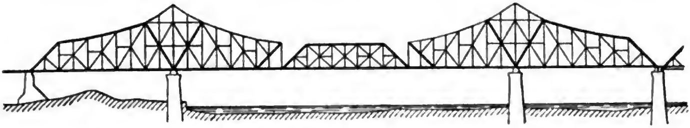


FIG. 25*i*. Layout of Cantilever Bridge with Idle Top Chord Members Omitted.

First. The anchor-bars should be made as long and as narrow as practicable, and should be divided into short lengths by pins, for the same reason as was given in the case of the suspenders of the suspended span.

Second. All anchorage details should be accessible to the paint brush, excepting, of course, those portions of the bottom girders which are buried in the masonry. This result is accomplished by leaving wells in the anchorages of sufficient size to permit the passage of a man to do the painting. If these wells are at any time partially filled with water temporarily by the rise of the stream, no harm will be done, provided that the painting of the metalwork therein be always attended to properly, and that adequate weep-holes be left in the masonry.

Third. Concrete for anchorages is always preferable to masonry, because it can readily be made to take any required form. If necessary, its exterior can be protected against abrasion from ice or drift by facing with granite or other hard rock, or with steel.

Fourth. There should be an independent anchorage against wind-pressure, obtained by sliding surfaces of steel, one of each pair of same forming part of a heavy detail which is rigidly attached to the bottom of the end floor-beam, and the other forming part of a heavy detail that is anchored firmly to the masonry.

Fifth. The tops of the anchor-piers should be made absolutely water-tight without interfering with the longitudinal expansion of the anchor-arm, so as to prevent rusting of the interior metalwork.

Sixth. The net weight of masonry in any anchor-pier after deducting the greatest buoyant effort of the displaced water should be twice as great as the maximum uplift on the said anchor-pier, when the effect of impact is duly included.

Many expedients have been used to connect up the metalwork of the suspended spans of cantilever bridges; and considerable trouble has often been experienced in doing the work, owing to variations in both length and elevation. The author is of the opinion that but little difficulty will be experienced if the following precautions be taken:

First. See that the entire triangulation is so accurately done that there will be no possibility of an error exceeding one-quarter of an inch in the distance between centres of pins over main piers. A perusal of Chapter LX will show that this is perfectly feasible.

Second. See that extra precautions are taken by the inspectors during the manufacture of the metalwork to insure that all lengths of main members shall be absolutely correct.

Third. See that the tapes used in shop and field are of exactly the same length.

Fourth. Use toggles like those described in this chapter for effecting the adjustment.

Fifth. Arrange to have the meeting ends of the chords a trifle high, so that lowering and not raising will be necessary.

Sixth. Arrange matters so that when the ends of the metalwork come together they will be a trifle apart rather than tending to lap, for it is much easier to heat the chords slightly by suspending beneath them sheets of metal containing slow fires than it would be to cool them by packing ice around them in cloths.

In preparing the manuscript of *De Pontibus*, the author made an elaborate series of calculations concerning the economics of cantilever designing and established curves for determining the weights of metal for all ordinary kinds of cantilever bridges. The principal problems solved were the following:

First: The ratio of the economic length of suspended span to that of the total opening.

Second. The most economic length of anchor-arms when the total length between centres of anchorages is given, and when the main piers can be placed wherever desired.

Third. The relations between the weights of metal in the suspended span, cantilever-arms, anchor-arms, anchorages, main pedestals, and anchor-spans.

Fourth. The best proportionate length for anchor-spans and the comparative weights of metal in those of different lengths.

The method of determining the economic functions was to take the data on hand for the before-mentioned Japanese cantilever bridges, exact weights of metal having been computed for structures of 320 ft., 400 ft.

and 500 ft. openings; and by varying the layouts so as to use longer and shorter suspended spans and longer and shorter anchor-arms, to obtain, by actual designs and estimates, the weights of metal for a sufficient number of layouts to indicate the desired minima.

In determining the economic length of suspended span for a certain opening, the length for the anchor-arms was first assumed to be one-fourth of the said opening, then the total weight of metal in the entire bridge, including even the anchorages and pedestals, was figured for several cases; and the length of suspended span giving the least weight of metal for the whole structure was found to be about three-eighths of the opening, although this length showed only one and a half per cent advantage over the case where the ratio was one-half. Now, as the rigidity of the entire structure certainly increases with the length of the suspended span, it will often be found best to make the length of the latter about one-half of the opening rather than three-eighths or any smaller proportion. On the other hand, though, it has been found by trial that, with the three-eighths ratio, there results a more sightly layout than can be obtained with the one-half ratio. To some extent the economic length of the suspended span will depend upon its method of erection, especially if it be pin-connected; for it is evident that, as its trusses must have some of their members either stiffened or strengthened in order properly to resist the erection stresses caused by cantilevering, a central span that is floated into place will weigh less than one of the same length which is cantilevered; hence, for economic adjustment of weights and lengths the floated span would have to be somewhat longer than the cantilevered one—but not much, as the difference in weights of the suspended trusses is probably never more than ten per cent.

Next there were tabulated the various component truss and lateral weights of a number of cantilever bridges that had previously been computed, and from them were constructed the curves shown in Fig. 25*j*, from which can be found the total weight of metal in the trusses and lateral systems of any three-span cantilever bridge, when the weight per lineal foot of the trusses and laterals in the suspended span is known. This weight, on the average, is eight per cent greater than that for an ordinary simple span of the same length, the extra metal being required mainly for stiffening certain truss members to resist erection stresses. Of course, if falsework be used for the suspended span, the eight per cent excess will not be added.

The curves of percentages are based on two assumptions, viz., first, the panels throughout the entire structure are of equal length, and, second, the lengths of the cantilever-arms and anchor-arms are the same. The first assumption is nearly always correct, for there is seldom any material advantage to be gained by varying the panel lengths in the various portions of the bridge. If the lengths of cantilever- and anchor-arms are unequal, the average weight of metal obtained for the latter by use of the curve will have to be corrected by the formula,

$$T' = \frac{T}{2}(1 + r)$$

where T' is the correct final weight of truss and lateral metal in the anchor-arm, T is the weight of same found by the percentage curve, and r is the ratio of length of cantilever-arm to that of anchor-arm. It should be observed that, in applying the percentage curves to structures having subdivided panels like those of the Petit truss, the main or double panel is to be used as the basis of calculation.

The method of applying the percentage curves is as follows: Let us take any opening and assume that there are six panels in each cantilever-arm, and that the weight per foot of truss and lateral metal in the suspended span is w, the panel length being p, and $pw = W$. It is to be observed that this method is applicable for any proportionate length of suspended span.

The weight of metal in the floor system, being independent of the span length and simply a function of the panel length and of the distance between trusses, is not considered in the investigation, but is, of course, to be added when figuring the total weight of metal in the structure.

The weight of truss and lateral metal in the cantilever-arm, as shown by the curve in Fig. 25*j*, will be

$$1.2\ W + 1.4\ W + 1.65\ W + 2.0\ W + 2.4\ W + 3.0\ W = 11.65\ W$$

The weight of the metal in the panel over the pier is, according to the directions on the diagram,

$$1.8 \times 3.0\ W = 5.4\ W$$

Let us assume that there are only five panels in the anchor-arm, then the trial weight T will be

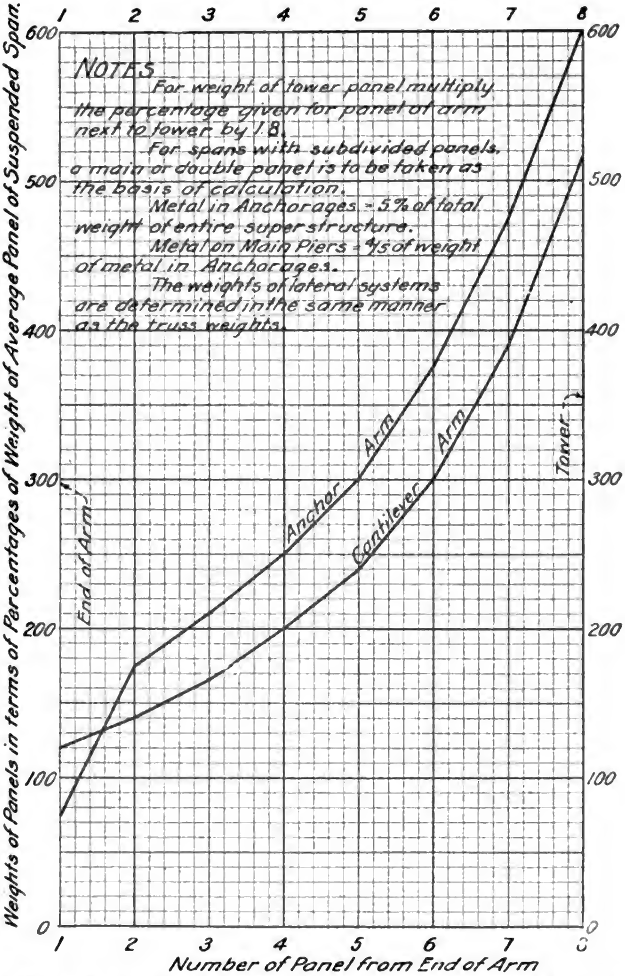
$$0.75\ W + 1.75\ W + 2.1\ W + 2.5\ W + 3.0\ W = 10.10W.$$

Substituting in the formula gives

$$T' = \frac{10.10\ W}{2}\left(1 + \frac{6}{5}\right) = 11.11\ W$$

It will be seen from these calculations that the full percentages given for the end panel-points of cantilever-arms and anchor-arms are to be used, although in reality there is but a half panel length for each point. This is caused by the heavy details required at these points for adjustment and anchorage. All erection metal at the end of a suspended span is assumed to belong to the cantilever-arm.

Should, in any case, the panel lengths be unequal in different portions of the structure, it will be a simple matter to use the curves by finding average weights per foot for two assumed cases of equal panel lengths, one making the arm greater and the other making it less in length than it actually is, and interpolating properly between the results for the required average weight per foot for the arm.



NOTES

For weight of tower panel multiply the percentage given for panel of arm next to tower by 1.8.

For spans with subdivided panels, a main or double panel is to be taken as the basis of calculation.

Metal in Anchorages = 5% of total weight of entire superstructure.

Metal on Main Piers = 4/5 of weight of metal in Anchorages.

The weights of lateral systems are determined in the same manner as the truss weights.

FIG. 25*j*. Weights of Trusses and Lateral Systems of Cantilever-arms and Anchor-arms in Percentages of Average Truss and Lateral Weights for One Panel of Suspended Span.

The total weight of metal in the two anchorages of any three-span cantilever bridge can be taken as five per cent of the grand total weight of metal in the said three spans, and the weight of metal in the pedestals on main piers at four per cent of same. Of course, conditions vary for different cases; nevertheless these percentages will give results sufficiently close for all practical purposes.

If the bridge be so long as to require an anchor-span, its weight of truss and lateral metal per lineal foot will be about 3.25 w, irrespective, strange to say, of the length of the said anchor-span, w being the weight per foot of the trusses and laterals in a suspended span, the length of which is three-eighths of the main opening. The explanation of this fact is that the weight per foot of the chords increases with the *net* bending moments, the upward moment being independent of the span length and affected only by the loads on the adjacent spans, but the downward moment increasing as the square of the span length; while the weight per foot of the web, in so far as it is affected by the shears from exterior loading (the ruling factor in determining the sections of the web members), varies inversely as the span length.

If the length of anchor-span be very short, say materially less than one-half of the main opening, the weight per foot for trusses and laterals will have to be increased to $3.5w$, notwithstanding the fact that the entire top chords may then be built of eye-bars; but such short spans would probably be barred out by consideration for navigation interests.

The percentage curves of Fig. 25j will not bear a rigid criticism, in that they make the weight of metal depend upon the number of panels. It is presupposed, however, that the panel length adopted is the most appropriate one for the bridge; and the curves will be found quite accurate whenever the proper panel length is used. With long panels the weight of metal per lineal foot found by the curves for cantilever- and anchor-arms is less than that found thereby for short panels. This is as it should be, but to a limited extent only; for it can be found by trial that an abnormally short or abnormally long panel length will give results too great or too small when checked by computations of weights made from actual designs.

These percentage curves were utilized in solving the next problem, viz., given the total distance between centres of anchorages and *carte blanche* as to the location of the main piers, to determine the length of each anchor-arm which will make the total weight of metal in the structure a minimum. This length was found to be two-tenths of the total distance between the anchorages.

It must not be forgotten that for every dollar saved by reducing the total weight of metal through the shortening of the anchor-arm, it will be necessary to spend about twenty cents for extra concrete in the anchorages. On this account, for the conditions assumed, the truly economic length of each anchor-arm of a three-span cantilever will generally be a little greater than twenty per cent of the total distance between centres of anchorages.

When, however, the problem is to determine the economic length of anchor-arm for a fixed distance between main piers, the result will be quite different; because, within reasonable limits, the shorter the anchor-arm the smaller will be its total weight of metal, and because trestle approach is much less expensive than anchor-arm. It would not, for evident reasons, be advisable to make the length of anchor-arm less than twenty per cent of that of the main opening, or say fifteen per cent of total distance between centres of anchorages. With this length there would probably be no reversion of stress in the chords of the anchor-arm, even when impact is considered. Generally, though, the appearance of the structure will be improved by using longer anchor-arms than the inferior limit just suggested.

In respect to the best proportionate length of anchor-spans, the latter weigh so much per lineal foot for all cases that the shorter they are made the greater the economy; but, as before stated, it is improbable that navigation interests would ever permit of their being made shorter than one-half of the main openings.

From the curves given in Fig. 25*j* and from other data, some of which were on file in the author's office and some of which were prepared especially for the purpose, were plotted the curves of weights of cantilever bridges given in Figs. 55*bbb* to 55*mmm* inclusive. From these can be found the average weights of metal per lineal foot of span for double-track cantilever bridges proportioned to carry certain live loads according to the specifications of Chapter LXXVIII. These weights were determined for the average of the total length of structure, and will not apply to any particular span of the layout. The proportions of span lengths assumed were as follows:

If l is the length of the main opening that is covered by one suspended span and two cantilever-arms, the length of the suspended span is three-eighths ($3/8$) of l, and that of each cantilever-arm and of each anchor-arm is five sixteenths ($5/16$) of l. If there be an anchor-span in the structure, its length is assumed to be five-eighths ($5/8$) of l. These proportions are fairly economic and may be found suitable for any particular crossing; but, if not, any small variation from them will cause no serious error in the weights of metal given by the diagrams.

As shown by the sketches in Fig. 55*aaa*, there are four possible layouts of spans for crossings where cantilever construction is adopted. The first is the one ordinarily employed. It consists of one central opening having a suspended span and two cantilever-arms, and two anchor-arms. The second consists of an anchor-arm, a cantilever-arm, a suspended span, another cantilever-arm, an anchor-span, another cantilever-arm, another suspended span, another cantilever-arm, and another anchor-arm; and for very wide crossings it can be lengthened by the addition of sections composed of one suspended span, two cantilever-arms, and an anchor-span, thus giving such a layout as that used for the Poughkeepsie Bridge (Fig. 25*y*). The third consists of a simple span resting on a pier at one end and suspended at the other, a cantilever-arm, an anchor-span, another cantilever-arm, and another simple span suspended at one end and supported by a pier at the other; and for long structures it can be extended by the addition of sections composed of one suspended span, two cantilever-arms, and an anchor-span, thus giving an arrangement such as was used for the Thebes Bridge (Fig. 25*s*). The fourth is a combination of the other types, and consists of a simple span resting on a pier at one end and suspended at the other, a cantilever-arm, an anchor-span, another cantilever-arm, a suspended span, another cantilever-arm, and an anchor-arm. These are all the possible legitimate layouts for any wholly cantilevered bridge. Of course, it is practicable to omit the suspended spans and connect to each other in a vertical direction, but not horizontally, the meeting ends of the cantilever arms; but such construction is unscientific, uneconomic, and exceedingly faulty, in that the stresses are rendered indeterminate except by making assumptions which are only approximately correct. Besides, the work involved in finding such stresses is complicated and excessive.

The Blackwell's Island Bridge over the East River at New York City, shown in Fig. 25*l*, is of this type; and after completion it was deemed so unsatisfactory that the authorities had the stresses refigured at enormous expense by independent computers, with the result that the overstresses were found to be so great (due to both ambiguity of stress distribution and overrun of dead load) that some of the roadways had to be omitted. A New York engineer connected with the bridge once remarked that the structure is so complicated that, if a man were to stand at the first panel point of the farthest span and were to spit into the river, his doing so would affect the stress in every main truss member of every span in the entire structure—and the statement is actually correct. The layout of this bridge is what may be termed a constructive lie. The top chords of the long spans were made into a continuous curve to resemble the cables of a suspension bridge, the object being æsthetics; but the attempt thus to beautify the structure was a failure, and the damage done to the bridge by the omission of the suspended span is measured by millions of dollars. This is a good illustration of the ill effects of violating Principle No. 1 of "The First Principles of Designing," given in Chapter XV, viz., "Simplicity is one of the highest attributes of good designing." No more effective example of its correctness than this structure affords could be desired.

Figs. 55*bbb* to 55*mmm*, inclusive, give curves of weights of metal for bridges of Types A, B, C, and D. When additional portions are added to Types B or D, the weight per lineal foot of these added sections will be ten per cent greater than that given by the curves of Type C. When additional portions are added to Type C, the weight per lineal foot of these added sections will also be ten per cent greater than that for a Type C bridge; and if there be n such portions added, the average weight of the entire bridge per lineal foot will be equal to that given by the curves for Type C, multiplied by the quantity $1 + 0.1 \frac{n}{n+1}$.

Each of the four types has characteristic features which adapt them to certain locations. They will now be discussed in detail.

Type A is best suited for a location at which a very long central span is required, while the side spans can be very short. As compared with a simple-span structure having the same central opening, and side spans of the same length as the anchor-arms, it will be found that the weights of metal in the two layouts will be equal when the length of the central span is about six hundred feet; while if trestle approaches can be used with the long simple span, the equality of weights will occur with a length of about seven hundred feet. The above figures are true for railway spans, the corresponding values for highway spans being somewhat smaller. If the use of falsework be impracticable, it will, of course, be advisable to employ the cantilever for lengths shorter than the limits just given; for the simple-span structure, if erected by semi-cantilevering, will be increased in weight by several per cent. Furthermore, with trestles or very short approach spans semi-cantilevering is generally impossible. If, however, the distance between end piers is a fixed quantity, and the two intermediate piers can be placed where desired, Type A will be found to require more metal than a layout composed of three simple spans of equal length erected on falsework, even when the lengths of the said simple spans are as great as one thousand feet. If the central simple span must be erected by semi-cantilevering, an equality of weights will occur when the lengths of the simple spans are about eight hundred feet, the corresponding length of the main opening of the cantilever layout being fifteen hundred feet. In many cases, however, the piers of the cantilever structure will be much cheaper than those of the simple-span bridge, on account of their being nearer to the banks of the river. For such a crossing, the total cost of the simple-span structure can sometimes be reduced by lengthening the centre span and shortening the side spans; and the most economic layout for the simple-span bridge should first be found, and its total cost then compared with that of the cantilever structure.

For a crossing where three spans of practically equal length can be employed, Type C will frequently prove economical. When the distance between the end piers exceeds nineteen hundred or two thousand feet, it will be found to require less metal than a bridge composed of three simple spans of equal length. It is not well adapted to crossings where falsework cannot be employed; and the use of the simple-span layout, in which the central span can be cantilevered out from the side ones, is frequently preferable on this account. Also, the adoption of the three duplicate spans will usually cause a reduction in the pound price of the metal.

Comparing Types A and C for a crossing in which the over-all length is fixed, but where the intermediate piers can be placed as desired, the ratio of the weight of Type C to that of Type A varies from about 0.8 for structures under two thousand feet in length to about 0.65 for structures three thousand feet long.

For an unusual crossing such as that found at the Blackwell's Island Bridge (Fig. 25*z*), in which there are two wide channels separated by a small island, Type B will be found to be the best of the cantilever layouts. Its cost is to be compared with that of a simple-span structure. The span length at which the weights of the two types of structure are equal will depend upon the construction required for the central portion of the simple-span structure, and also upon the relative lengths of the main openings and anchor-span in the cantilever bridge; but ordinarily it will be about six or seven hundred feet. Where falsework cannot be employed, the cantilever may be preferable for even shorter span lengths.

For a long crossing over a wide river, where the foundation conditions are uniform and the piers can be located as desired, Type C, with as many added sections as are necessary, will be the most economic of the cantilever layouts. As compared with a simple-span layout of five equal spans erected on falsework, it will require less metal when the span lengths of the latter exceed six hundred feet; and if the simple spans are designed for erection by semi-cantilevering, this limit becomes about five hundred and fifty feet. The adoption of the simple-span layout will give several duplicate spans, which will reduce slightly the pound price of the metal-work. So far as erection is concerned, there is but little choice between the two structures; for in either layout alternate spans must be erected on falsework, while the intervening ones can be erected wholly or partly by cantilevering.

A special case arises in such a crossing as that at Poughkeepsie (Fig. 25*y*), where the end piers could be built much more cheaply than those in the stream. The layout adopted was Type B. Had there been used a Type C structure giving five openings between the end main piers, and two hundred feet of approach at each end instead of the anchor-arms, the total weight of the superstructure would have been increased about two per cent. For most crossings, the cost of the anchor piers in the layout used would have overbalanced the small saving in the superstructure, thus making the Type C structure cheaper; but in this instance the cost of the anchor piers was comparatively small, and the layout employed is, therefore, the more economic of the two. A Type C structure giving seven openings between the anchorage piers could also have been used, thus effecting a reduction of about five per cent in the weight of the superstructure metal; but this would have increased decidedly the cost of the piers next to the end ones (which would have then been located in the stream instead of on the banks), and would doubtless have more than offset the saving in the superstructure metal. It is evident, therefore, that in some cases Type B should be employed for long crossings. The necessity for providing long openings next to the banks of a stream might also require its adoption. Should such special conditions occur at one end only of a bridge, Type D would naturally be employed. This was done in the case of the Memphis Bridge (Fig. 25*o*).

In regard to truss depths for cantilever bridges, the author's practice is to make that for the suspended span, when the chords are parallel, from one-fifth of its length for short spans to one-seventh of its length for very long ones, interpolating between these limits for intermediate lengths. If one of the chords be polygonal, a greater proportionate truss depth at mid-span and a smaller one at the ends would logically be employed. The height of the vertical posts over the main piers can be made about fifteen (15) per cent of the length of the main opening, or not to exceed three and a half (3.5) times the perpendicular distance between central planes of trusses over the main piers. In the new design for the Quebec bridge these posts were made 310 feet high for the sake of appearance, although the economic length was found to be only 290 feet. These figures correspond to percentages of main openings of about seventeen (17) and sixteen (16) respectively.

For the sake of appearance the centres of the top chord pins in cantilever-arms are best placed on arcs of parabolas, the vertices of which are located at the hips of the suspended span; and the anchor-arms are laid out to the same curve, beginning at the tops of the posts over the main piers.

The limiting length of main opening for cantilever bridges is treated fully in Chapter IV.

There are certain legitimate economies that may be employed in the designing of cantilever bridges, among which may be mentioned the following:

A. The wind pressure assumed in computing the erection stresses may be taken lower than that given in the specifications for the finished structure, provided that the full wind pressure would not overstress any of the metal seriously or involve any risk of disaster during erection. A stress of three-quarters of the elastic limit of the metal applied a few times during erection would do no harm, and the chance of there being in that limited time any wind pressure at all approaching in magnitude that specified is very small. This lowering of the intensity of wind pressure may be the means of avoiding, in a perfectly legitimate manner, the increasing of the sections of a number of truss members because of erection stresses; but such economizing should be done with caution after a thorough consideration of its greatest possible effects.

B. A certain amount of metal can sometimes be saved by splaying the trusses between the main piers and the ends of the cantilever and anchor arms; but unless the amount thereof be fairly large, the extra pound price of the metalwork in the cantilever- and anchor-arms due to the said splaying may more than offset the value of the reduction.

C. A small economy may sometimes be accomplished by omitting during erection from the cantilevered portion of the structure all parts that are not essential to its strength before the coupling of the cantilevered ends is effected, thus reducing the erection stresses a little.

D. Solitary piers or large pedestals under the main vertical posts are sometimes just as satisfactory in every way as long, continuous piers, especially if a connecting wall of reinforced concrete between them be employed. Generally they will be found to involve a large saving in the cost of the substructure.

E. In very wide cantilever bridges it might sometimes be advisable to adopt intermediate trusses so as to economize materially in the weight of the floor-beams and a trifle in that of the trusses, also because of the consequent reduction in dead load, but mainly so as to keep within reasonable limits the sizes and weights of the pieces to be handled and thus economize on the size of the traveler and the cost of the erecting machinery. On the other hand, though, increasing the number of trusses is likely to increase a little the percentage of weight of truss details; but where the sections of members are large this increase would be small. In case the wind stresses are an important factor in the proportioning of the truss members, the employment of an interior truss or interior trusses might, by the reduction in areas of chord sections, cause such relatively large wind stresses on the chords of the exterior trusses that the additional metal required to take care of them would offset all the saving obtained in the ways just mentioned.

F. In long-span cantilever bridges the stresses on the truss members that rest upon the piers should be divided among as many such members as possible by using an inclined strut on each side as well as a vertical post instead of carrying all the loads to the top of the latter by tension members, as was done in the design of the ill-fated Quebec bridge. Again, if a lowering of the inner ends of the cantilever arms be permissible, the inclining of the end sections of the bottom chords to the horizontal will take up a portion of the load that is carried to the pier and thus will reduce the stresses on the vertical and inclined posts assembling there. This last feature reduces also the total cost of the masonry by diminishing the height of the main piers, and saves placing the tops of the trusses at an abnormal height above the water.

G. If there be any choice between the riveted and the pin-connected types of construction for any cantilever bridge, it is generally better to adopt the latter, because, as cantilever bridges are usually employed for long spans only, pin-connected work is the more suitable. Again, it is a little lighter than riveted work and therefore the dead load on the structure would be somewhat less. On the other hand, the riveted construction is so much more rigid than the pin-connected that it is preferable to adopt it whenever the conditions permit; besides, in the riveted work it is not necessary to stiffen any truss members for erection, although it might be obligatory to increase a few of their sectional areas.

H. Very large compression members should be made of box section so as to do away with latticing. This not only effects an improvement in the design, but also saves some metal, although the details required at the panel points to distribute the stresses from the cut cover plates tend to offset the saving in weight of lattice bars and stay plates.

Chapter III of Merriman and Jacoby's "Roofs and Bridges," Part IV, presents an excellent treatment of the subject of cantilever bridges, discussed mainly from the theoretical point of view; but it gives also something concerning the history of cantilever bridge building and a list of the principal cantilever bridges of America. The professors' economic investigations, which are based upon chord weights only, show that for a three-span cantilever of Type A, each anchor-arm should be about twenty-one and two tenths (21.2) per cent of the total length of structure. This is quite a close agreement with the twenty (20) per cent found by the more accurate and practical investigation that was made for *De Pontibus*. The professors find, though, thirty-nine and four tenths (39.4) per cent of the total length of structure for the economic length of the suspended span, corresponding to about sixty-eight per cent of that of the main opening, while the *De Pontibus* investigation made it only thirty-seven and a half (37.5) per cent. Actual experience has repeatedly shown that the economic length of the suspended span is from three-eighths (3/8) to one half (1/2) of the main opening, hence the professors' figures for this portion of their work have the appearance of being incorrect; but Prof. Merriman has explained to the author by letter that he assumed the truss depth to be the same throughout the entire structure. This assumption, combined with that of ignoring the effect of the weight of the web, will account for the large discrepancy; because the professors' mathematics have been checked and found to be faultless. As a matter of fact, though, no American engineer would think for an instant of making the truss depth constant throughout the structure, because for economic reasons it should generally be about twice as great over the main piers as in the suspended span. European engineers, however, often fail to make the truss depths, especially in the cantilever- and anchor-arms, great enough for economy.

The professors make also from their economic investigations the following deduction: "The cantilever system hence has no theoretic economy over simple trusses when the piers can be located in any position; moreover, when the influence of the alternating stresses in the anchor-arm and the material required for anchor rods are taken into account, it is at a marked disadvantage." As has been shown previously in this chapter, this statement is true for Type A, which is the layout employed by the professors; but it is not correct in general. The professors show also by their theoretical computations that the greatest deflection of a cantilever bridge of ordinary proportions is far more than that for a bridge of three equal spans having the same aggregate length; thus confirming the statement previously made in this chapter concerning the comparative rigidities of the two layouts considered.

The professors are right in their surmise that "probably the common three-span-cantilever bridge has a lower degree of economy than the arrangement where the simple trusses are in the end spans, as in the Kentucky River bridge"; for, as previously stated, Type C layout requires only from eighty to sixty-five per cent as much metal as does Type A, for the same total length of structure. It must be remembered, however, that, as previously indicated, the comparison is hardly fair to the common three-span-cantilever, because the latter provides a greater main opening than that of the alternative layout.

Shortly after the curves for weights of metal given in Fig. 25*j* were prepared, an excellent check on their accuracy was obtained from the published estimated weights for the longest span cantilever bridge ever designed, viz., 2,300 feet measured between centres of main piers. It was prepared by the Union Bridge Company for a proposed crossing of the North River at New York City.

The total weight of metal in trusses and laterals of the 720 ft. suspended span was 10,400,000 lbs. The trusses, which were of the Petit type, were divided into six main panels of 120 ft. each; consequently the panel weight was 10,400,000 ÷ 6 = 1,733,000 lbs. In the cantilever-arm there were six and five-eighths main panels; consequently the weight of trusses and laterals therefor would be

$$1.20\,W + 1.40\,W + 1.65\,W + 2.00\,W + 2.40\,W + 3.00\,W + \tfrac{5}{8} \times 3.60\,W = 13.90\,W = 24{,}090{,}000 \text{ lbs.}$$

Each anchor-arm was 840 ft. long and was divided into seven double panels, and there were seven and five-eighths loads to be considered; consequently the weight of trusses and laterals therefor would be

$$0.75\,W + 1.75\,W + 2.10\,W + 2.50\,W + 3.00\,W + 3.75\,W + 4.75\,W + \tfrac{5}{8} \times 5.75\,W = 22.19\,W = 38{,}455{,}000 \text{ lbs.}$$

This weight must be reduced owing to the fact that the length of the cantilever-arm is only six-sevenths of that of the anchor-arm, making $r = 0.857$.

$$T' = \frac{T}{2}(1 + r) = \frac{38{,}455{,}000}{2}(1.857) = 35{,}705{,}000 \text{ lbs.}$$

The total weight by the curves for the two cantilever- and anchor-arms is, therefore,

$$2\,(24{,}090{,}000 + 35{,}705{,}000) = 119{,}590{,}000 \text{ lbs.}$$

The total weight of metal given in the published estimate for trusses and laterals for the two cantilever- and anchor-arms, after deducting 11,500,000 lbs. for weight of metal in the anchorages and ignoring the allowance for sundries (which was properly put in for prudential reasons) was 119,700,000 lbs., making the difference 110,000 lbs., or about one-tenth of one per cent.

About the same time as these data concerning the North River Bridge were published, the author, in making the calculations for his preliminary study of the Quebec crossing of the St. Lawrence River, obtained a partial check on the curves of Fig. 25*j*, for he computed in detail the weights of metal in the suspended span and the cantilever-arm, estimating the remaining weights from the curves. The error found for the cantilever-arm curve was only one eighth of one per cent.

In the design prepared some years ago by the author for a cantilever bridge with a main opening of 1,830 feet for a proposed crossing of the Strait of Canso between Cape Breton Island and the mainland of Nova Scotia, Fig. 526, another close check on the curves was obtained.

These extremely accurate checks might lead one to believe that the curves are absolutely reliable for all layouts and conditions; but such is not the case, because since they were published in *De Pontibus* the author has had occasion to make a number of tests, some of which varied as much as three or four per cent, but these great variations were caused by peculiarities in the panel lengths adopted and by variation in the type of truss between the suspended span and the rest of the structure. At any rate, the reader may rest assured that these curves and those given in Figs. 55*bbb* to 55*mmm*, inclusive, give the most accurate data yet published concerning the weights of cantilever bridges.

Before passing to the subject of details for cantilever spans, some observations will be made concerning a number of the largest and most important cantilever bridges yet built, arranged in the order of the relative lengths of their main openings.

The longest span cantilever bridge which has ever been attempted is the one across the St. Lawrence River at Quebec, which failed a few years ago, and which is now being rebuilt on a different layout, as described later in this chapter, and from much better specifications.

The largest cantilever bridge yet built is the one at Queensferry over the Firth of Forth (Fig. 25*k*), the main portion of which consists of two spans of 1,710 feet each, with central spans of 350 ft. each and two anchor-arms of 690 ft. each. The length of the tower-span over the centre piers is 260 ft., and that of each of the two other tower-spans is 145 ft., making the total length of the main structure 5,330 ft. The design for this bridge and a complete history of its construction are given in a special work published by *Engineering* (London).

The exceptions which the author would take to this design are as follows:

First. The suspended spans are just about one half as long as they ought to be for both appearance and economy.

Second. The structure should have been made pin-connected for both ease of erection and certainty of stress distribution.

Third. A single system of cancellation for the webs of the girders would have been more scientific than the double system adopted, and would not have been any more expensive.

Fourth. The structure, as a whole, from the point of view of American engineers, was unnecessarily expensive.

On the other hand, though, the labor involved in both the designing and building of this bridge was immense; and the successful completion of the structure is a great credit to all concerned in its designing and construction.

The cantilever bridge having the next longest span is the Blackwell's Island Bridge (Fig. 25*l*) over the East River in New York City. Starting at the west end, the spans are as follows: an anchor arm of 470 ft., a span of two cantilever-arms without a suspended span, giving an opening of 1,182 feet, an anchor span of 630 ft., a span of two cantilever-arms without a suspended span, giving an opening of 984 ft., and an anchor-arm of 459 ft., making a total length of bridge proper of 3,725 ft. This structure has been mentioned before in this chapter and has been criticized for the falseness of its lines and for the folly of omitting the suspended spans from the main openings, thus complicating almost beyond comprehension the computation of stresses in the trusses. It was bad policy to carry all the load by tension members to the top of the vertical post over each pier instead of using inclined struts and thus dividing the load between three compression members. The dead loads in this structure were under-estimated to such an extent that serious overstresses exist in the trusses. So large were they that, as before stated, it was decided to reduce the total load by not putting in some of the roadways which the bridge was originally intended to carry.

The cantilever bridge with the next longest main opening is the Landsdowne Bridge (Fig. 25*m*) over the Indus River at Sukkur, India. It consists of a single span of 820 feet without anchor-arms (the latter being replaced by guys). It has a suspended span of 200 feet. The appearance of this bridge is bizarre in the extreme, and the structure is economic in neither weight of material nor cost of shopwork. Compared with an American bridge of the same span, capacity, and strength, the weights of metal in the 820 foot span, only, would be about in the ratio of 1.33 to unity.

Next on the list comes the Monongahela River Bridge (Fig. 25*n*) of the Wabash Railroad at Pittsburg, Pa., with its main opening of 812 feet and two anchor-arms of 346 feet each, making a total length of 1,504 feet between centres of anchorages. This is a double-track railway structure and is built to carry heavy live loads. The layout is of pleasing appear-

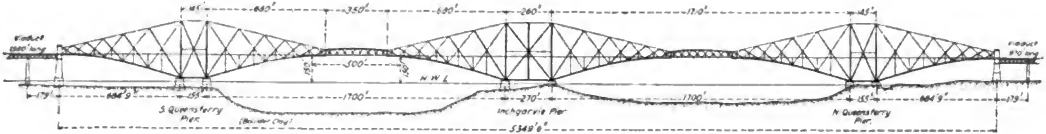


Fig. 25*k*. Bridge over the Firth of Forth at Queensferry, Scotland.

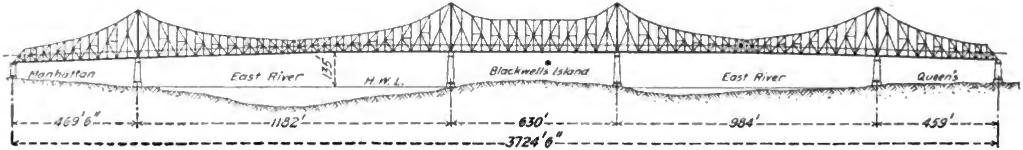


Fig. 25*l*. Blackwell's Island Bridge over the East River in New York City.

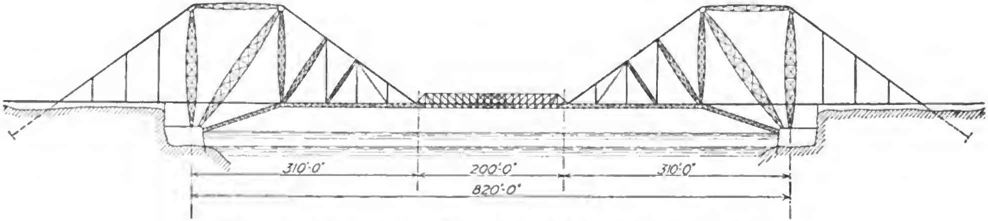


Fig. 25*m*. Landsdowne Bridge over the Indus River at Sukkur, India.

ance. The top chords of the suspended span are horizontal and those of the cantilever-arms and anchor-arms are polygonal, the apices being located on sightly curves. The main vertical posts are double with short, horizontal struts, but no diagonals, between their component parts. The length of the suspended span is 360 feet or about forty-four (44) per cent of the total main opening, which is quite economical. The ratio of length of anchor-arm to total length of structure is 0.23, which is but little greater than the truly economic length. The distance between central planes of trusses is thirty-two (32) feet or about one twenty-fifth (1/25) of the span. The tower depth is one hundred and twenty-six and a half (126½) feet, or between fifteen (15) and sixteen (16) per cent of the length of the main opening. This bridge was designed by Messrs. Boller and Hodge, Consulting Engineers, of New York City.

The cantilever having the next longest span, viz., 790 feet, is the old railway bridge at Memphis over the Mississippi River. This structure is both unsightly and uneconomical of material. Its layout of spans is unfortunate (but the War Department, and not the designer, is responsible for this), and the truss depths are far too small for both economy and appearance (see Fig. 25*o*). For the latter fault, however, the War Department cannot be blamed. The validity of this criticism is evidenced by the fact that the truss depth of the new bridge (see Fig. 25*o*) is a little over ten feet greater than that of the old structure, while the principal span lengths remain the same.

The bridge with the next longest opening is the one across the Ohio River at Beaver, Pa., on the line of the Pittsburg and Lake Erie Railroad. It has two anchor-arms of 320 feet each, two cantilever-arms of 242 feet each, and a suspended span of 285 feet, making the distance from centre to centre of main piers 769 feet. It was designed by Albert Lucius, Esq., Consulting Engineer, and was built by the McClintic-Marshall Construction Company under the direction of Paul L. Wolfel, Esq., Chief Engineer. As can be seen from Fig. 25*p*, the appearance of the structure is most æsthetic.

Next comes the Sewickley Bridge across the Ohio River. It consists of two anchor-arms of 300 feet each, two cantilever-arms of 200 feet each, and a suspended span of 350 feet, making the main opening 750 feet. It is a highway structure and one of the longest of that class in the cantilever type that has yet been constructed. As can be seen from Fig. 25*q*, its outline is pleasing and its layout is almost perfectly symmetrical about the central vertical transverse plane. The span lengths, panel lengths, and truss depths adopted show that economy as well as appearance was studied when the layout was prepared. It appears that there were three engineers responsible for the design, viz., A. G. Chalfant, Esq., County Engineer of Allegheny County, G. Gudmundsson, Esq., Consulting Engineer, and A. W. Buel, Esq., Consulting Engineer.

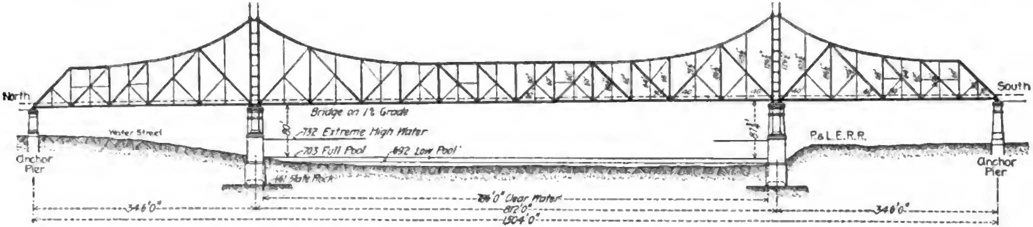
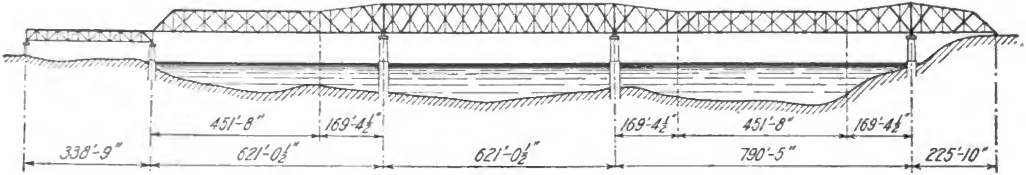
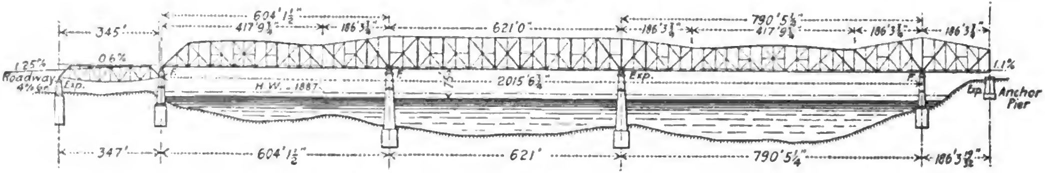


Fig. 25*n*. Wabash Railroad Bridge over the Monongahela River at Pittsburg, Pa.



Old Bridge.



New Bridge.

Fig. 25*o*. Railway Bridges over the Mississippi River at Memphis, Tenn.

Next in order is the Ohio River bridge at Mingo Junction (Fig. 25*r*) designed and engineered by Messrs. Boller and Hodge for the Wabash Railroad Company. It is laid out on similar lines to those of their Monongahela River bridge and presents the same graceful appearance. Its central opening is 700 feet.

The next on the list is the Thebes bridge (Fig. 25*s*) over the Mississippi River. Beginning at the west end, the openings are as follows: a simple span and a cantilever-arm aggregating 518 feet, an anchor-span of 521 feet, a main opening of two cantilever-arms and a suspended span aggregating 671 feet, an anchor span of 521 feet, and a cantilever-arm and simple span aggregating 518 feet, in all about 2,750 feet. The approaches are of concrete arches. The bridge proper is perfectly symmetrical about its middle point, but the approaches differ somewhat in both total length and arch openings. The symmetry, of course, is pleasing, but the structure as a whole is too squat for fine appearance. As before stated in this chapter, the appearance could have been much improved by putting in five simple spans, all but the middle one being alike, and that being in outline simply an enlargement of the others. The engineers were Ralph Modjeski, Esq., of Chicago, and Alfred Noble, Esq., of New York City.

Next in size is a bridge over the Rhine at Ruhrort, Germany (Fig. 25*t*). It consists of a simple span of 221 feet and a cantilever-arm of 53 feet forming one opening, an anchor-span of 421 feet, two cantilever-arms of 112 feet each and a suspended span of 443 feet forming a main opening of 667 feet, an anchor-span of 399 feet, and a cantilever-arm of 53 feet and a simple span forming an opening of 239 feet, making the total length of structure 2,053 feet. There are no approaches. The outlines of the bridge are fairly good, but the truss depths throughout are far too small for both economy of metal and appearance. The towers are only 82 feet high, and the truss depths are less than one-tenth of the span lengths. The shore spans have pony trusses, but all other spans have through trusses. It is a highway structure thirty-six (36) feet wide between central planes of trusses and fifty-two and a half (52.5) feet wide from out to out.

Next comes the Red Rock Bridge (Fig. 25*u*) over the Colorado River on the Atlantic and Pacific Railway, now a branch of the Santa Fe System. It was designed some twenty-seven years ago by the author, and it has lately been remodelled, because the live loads that are liable to come on it are about seventy-five per cent greater than those for which it was figured. It consisted originally of a main span of 660 feet and two anchor-arms of 165 feet each, the length of the suspended span being 330 feet. The width between central planes of trusses is 25 feet, and the truss depth varies from 55 feet for the suspended span to 101 feet for the vertical posts over the main piers. The bridge was designed to meet certain conditions, economy in first cost being the prime requisite; con-

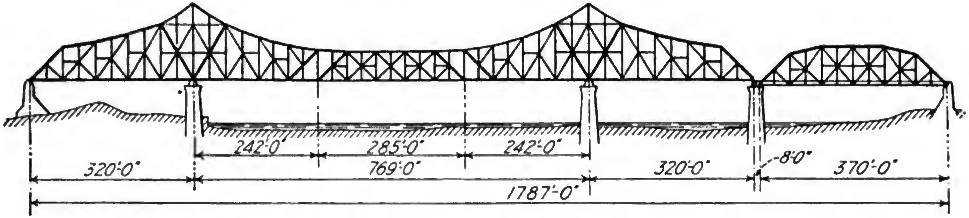


FIG. 25*p*. Pittsburg and Lake Erie Railroad Bridge over the Ohio River at Beaver, Pa.

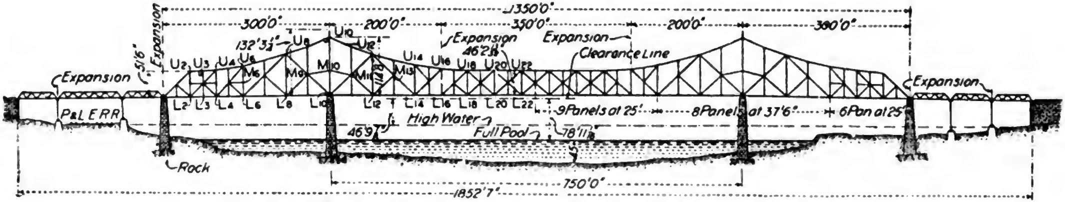


FIG. 25*q*. Highway Cantilever Bridge over the Ohio River at Sewickley, Pa.

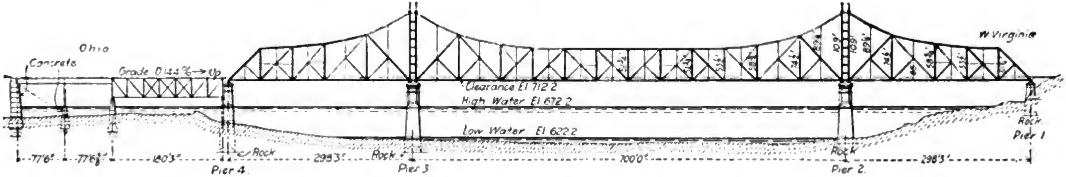


FIG. 25*r*. Wabash Railroad Bridge over the Ohio River at Mingo Junction, Ohio.

sequently the subject of æsthetics did not receive great consideration. Engineers and architects differ fundamentally in their opinions concerning the architectural effect in this structure. Some approve its appearance; others characterize it as harsh in its outlines. The relations between lengths of suspended span, cantilever-arms, and anchor-arms, and the matters of width and depth, although very hurriedly determined, have since been found to be just about the best practicable. This bridge is described very fully in the *Transactions* of the American Society of Civil Engineers for 1891.

The next in order is the Marietta bridge (Fig. 25*v*) over the Ohio River. It is a highway structure on a steep grade, and is irregular in its layout of spans. Its appearance is far from pleasing. There is one opening of 650 feet covered by a suspended span of 270 feet and two cantilever-arms (one of 80 feet and the other of 300 feet), also one anchor-arm of 130 feet and another of 600 feet. Besides, there are two simple spans of 220 feet each.

Next there comes the Cernavoda Bridge (Fig. 25*w*) over the Danube in Roumania. It consists of a simple span of 295 feet and a cantilever-arm of 164 feet forming an opening of 459 feet, an anchor-span of 459 feet, two cantilever-arms of 164 feet each and a suspended-span of 295 feet forming the main opening of 623 feet, an anchor-span of 459 feet, and a cantilever-arm of 164 feet with a simple span of 295 feet forming an opening of 459 feet. The trusses are of the double intersection triangular type, and the towers are accentuated by the adoption of a liberal height. Although the bridge may seem odd to the trained eyes of American engineers, its appearance is not altogether unpleasing, because the perfect symmetry of its entire layout is quite striking, as is shown in the cut.

Next is the Inter-provincial bridge (Fig. 25*x*) over the Ottawa River at Ottawa, Canada. It consists of one anchor-arm of 247 feet, one main span of 556 feet, one anchor-arm of 247 feet, one simple span of 247 feet, and another simple span of 140 feet. Barring the irregularity of the Quebec approach, the layout is rather pleasing. The suspended span is 309 feet long, which is too great for economy, and the main vertical posts are 90 feet long. The bridge carries wagon, street-railway, and pedestrian traffic.

The next on the list is the Poughkeepsie Bridge (Fig. 25*y*), a deck structure, which, besides the short approach spans, consists of a 201 foot anchor-arm, two cantilever-arms and a suspended span aggregating 548 feet, an anchor-span of 525 feet, two cantilever-arms and a suspended span aggregating 546 feet, an anchor-span of 525 feet, a cantilever-span like the last described, and an anchor-span like the one first mentioned. The masonry of each pier extends a few feet above extreme high water, and upon it is built a narrow steel braced tower with vertical legs. As before stated in this chapter, all these spans should have been simple

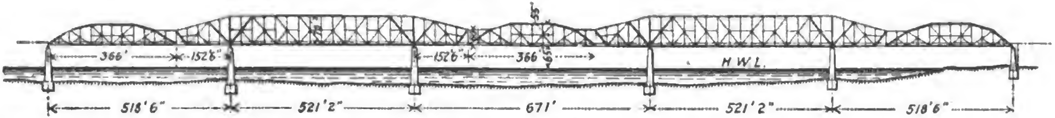


FIG. 25s. Railway Bridge over the Mississippi River at Thebes, Ill.

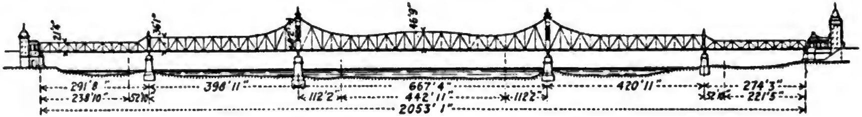


FIG. 25t. Bridge over the Rhine at Ruhrort, Germany.

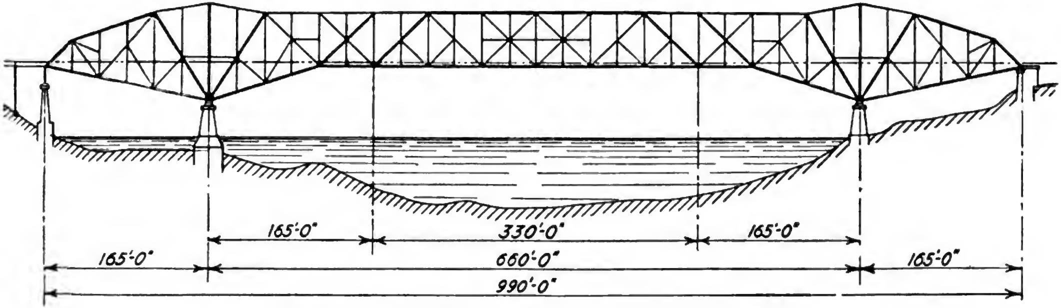


FIG. 25u. Red Rock Cantilever Bridge over the Colorado River.

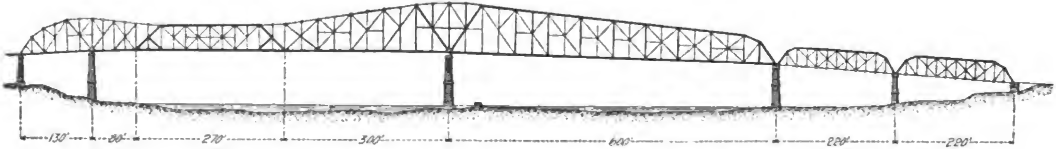


FIG. 25*v*. Highway Cantilever Bridge over the Ohio River at Marietta, Pa.

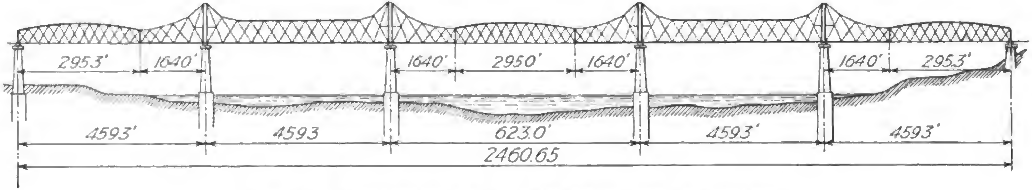


FIG. 25*w*. Cernavoda Bridge over the Danube River in Roumania.

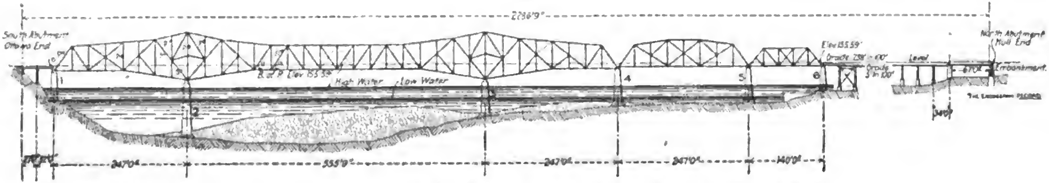


FIG. 25*x*. Interprovincial Bridge over the Ottawa River at Ottawa, Canada.

ones and of the same length. The resulting structure would then have been far more rigid and the weight of metal required would have been a little reduced. The steel towers, while possibly less expensive than masonry ones, compress much more under live load, and, consequently, are not as satisfactory. This bridge was built at a time when, on account of their novelty, cantilever bridges were quite popular, and when there was a prevalent impression in the public mind that they possessed some inherent virtues which rendered them superior to ordinary structures.

Next in order comes the Yellow River bridge near Tsinanfu on the line of the Tientsin-Pukow Railway in China. The design is the work of German engineers; and it shows their earmarks, for the trusses are of the sub-divided Warren type and their depths are abnormally small. The cantilever portion of the structure consists of two anchor-arms of 420 feet each, two cantilever-arms of 90 feet each, and a suspended span of 360 feet, making the length of the latter two-thirds of the main opening. It is claimed that this proportion "approximates to the proportion of greatest economy, and yet does not give an excessive length of suspended span." If the statement is correct (which the author thinks it is not), it is due to the fact that the truss depths over piers are far too small for economy. If they were increased, the total weight of metal would be lessened, and, consequently, it would be economic to lengthen the cantilever-arms. To the American engineer's eye the structure is too squat for æsthetics, as can be seen from Fig. 25*z*. The bridge is intended to carry ultimately two standard gauge railway tracks, but at present only one is laid, being placed in the middle with a footwalk on each side. The trusses are proportioned for only one track, and, when the other track is required, duplicate trusses are to be added outside of the present ones, suitably connected to the old trusses. As this method of providing for future double-tracking was evolved and patented by the author a dozen years ago in the United States and Canada, he feels that morally he has a mortgage on the Chinese structure; but, unfortunately, he did not adopt the precaution of taking out his patent in China.

Next in size comes the Long Lake Highway Bridge in Hamilton County, N. Y., with its span of 525 feet between centres of tower posts, the cantilever-arms and the suspended span all being of the same length, 175 feet. This bridge is of unusual design, for there are no anchor-arms, backstays running back to anchorages being used instead. The bridge is a very light highway structure built as cheaply as possible. The width between central planes of trusses in the suspended span is 16 feet, and at towers 24 feet. The suspended span was built and floated into position upon a raft made of kerosene oil barrels. It weighed only forty tons. The cost of erecting the metalwork of this bridge was only $15 a ton.

Next comes the Connel Ferry bridge (Fig. 25*aa*) which carries the Callender and Oban Railway across Loch Etive, Scotland. To American eyes this bridge has a peculiar appearance. Its effect is striking, and

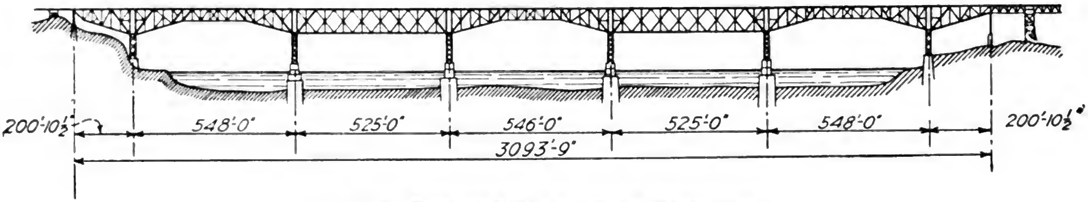


FIG. 25*y*. Poughkeepsie Bridge over the Hudson River.

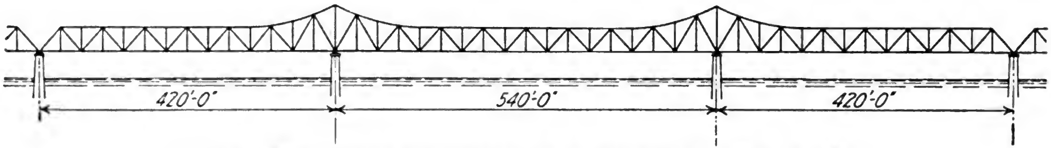


FIG. 25*z*. Tientsin-Pukow Railway Bridge over the Yellow River near Tsinanfu, China.

the perfect symmetry of the structure is gratifying; but its lines are too severe for beauty, and the economics of the design are worse than questionable. The design of the metalwork is of an unusual character. There are no vertical posts over the main piers, but instead there are large battered triangles with their apices projecting some fifty-four (54) feet beyond the centres of bearing and supporting the span by a hinged-joint. This makes the length of each anchor-arm about 159 feet, that of each cantilever-arm 146 feet, and that of the suspended span 232 feet. The splaying of the triangles, which was really unnecessary for stability, caused an extravagant use of masonry for the piers.

The next is the Cincinnati and Newport Highway bridge (Fig. 25*bb*) over the Ohio River. Besides the simple span in the approaches it contains two anchor-arms of 252 feet each and a main cantilever span of 520 feet, the length of the suspended portion being 208 feet, which is almost the economic length. The outlines of the cantilever portion of the structure are quite good, but the lack of symmetry in the approaches detracts from the general appearance.

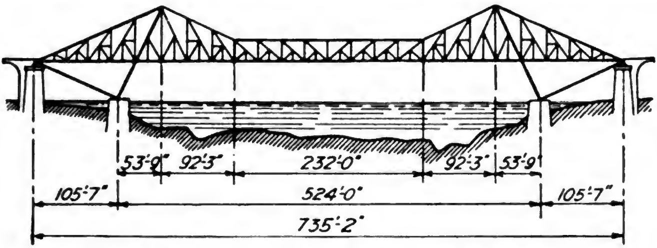


FIG. 25*aa*. Connel's Ferry Bridge Carrying the Callender and Oban Railway across Loch Etive, Scotland.

Finally on the list comes the Tyrone bridge (Fig. 25*cc*) over the Kentucky River on the line of the Southern Railway. It consists of two steel trestle approaches and a three-span cantilever having a main opening of 520 feet and anchor-arms of 208 feet each, besides two tower spans of about thirty feet each. The structure is quite symmetrical, but the trussing adopted is unusual and somewhat unsightly because of its irregularity.

Fig. 25*dd* shows a view of the new Quebec bridge (now under construction) designed by the engineers of the St. Lawrence Bridge Company, an organization founded solely for the building of this structure and composed of the Dominion Bridge Company of Montreal and the Canadian Bridge Company of Walkerville, Canada. The length of each anchor-arm is 515 feet, that of each cantilever-arm 580 feet, and that of the suspended span 640 feet, making the total length of main opening 1800 feet. While the layout of structure can scarcely be termed æsthetic,

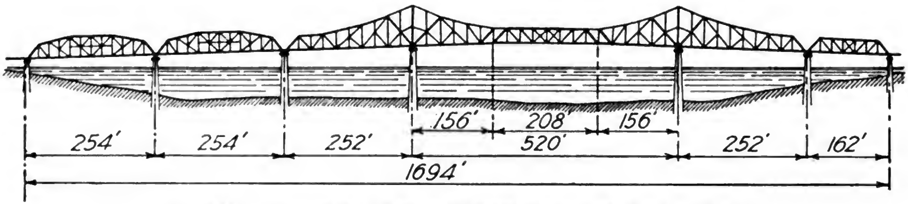


FIG. 25*bb*. Cincinnati and Newport Highway Bridge over the Ohio River.

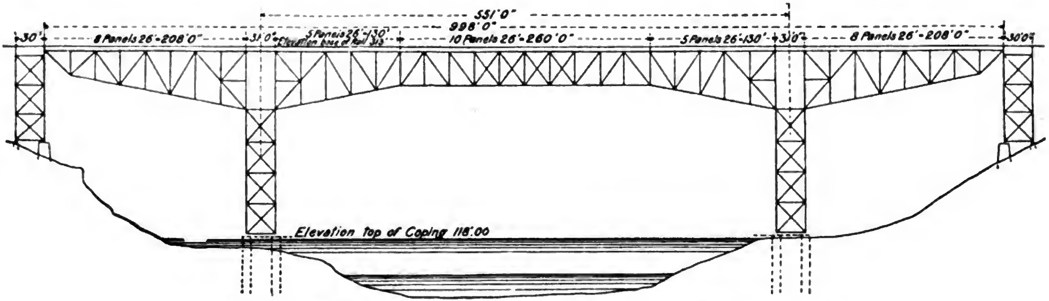


FIG. 25*cc*. Southern Railway Bridge over the Kentucky River at Tyrone, Ky.

its very massiveness and simplicity give it a rather pleasing appearance. As it is a contractor's design, its layout certainly ought to be economical; and it is well known that much study was devoted to it. The length of the suspended span is 34% of the main opening, which is in close accord with the economic percentage (38) determined by the author in 1896.

There are a number of new features of importance in this design, among which may be mentioned the floating of the suspended span into position, the *K* system of truss cancellation, the hinging of floor-beams at ends for the purpose of obtaining exactly central loading of the trusses and, consequently, the avoidance of secondary stresses from the connection, full splices for all compression members, the omission of lateral bracing in top-chord planes and the consequent forcing of the wind stresses to the lower lateral system, unprecedentedly large main pedestals, the banking of eye-bars in top chords,* the trussing of the eye-bars to uphold their weight between supporting points, the connection of web members by pins on gravity lines but eccentric to the panel-points, and the use of double floor-beams to connect to the main vertical posts.

The advantages claimed for the *K* truss by the designers of the new Quebec bridge are as follows:

a. Safety to life and property during erection, as well as economy and rapidity in construction.

b. Minimum number of secondary members, and few, if any, temporary members during erection.

c. The division of web stresses, thus reducing the sections of the web members to more practical dimensions and simplifying the details.

d. Economy of material.

e. Improved appearance.

f. Uniform deflection, reducing the secondary stresses.

The preceding examples, as far as the author

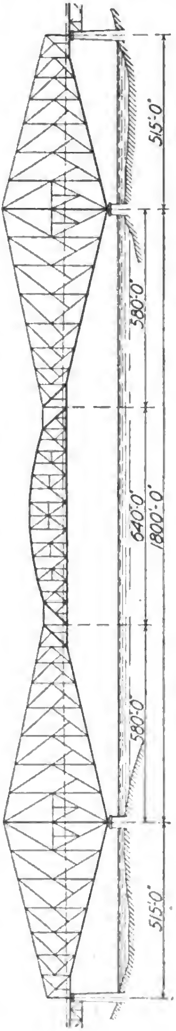


FIG. 25*dd*. Quebec Bridge over the St. Lawrence River.

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* This detail of construction was first employed by the late Joseph Tomlinson, Esq., C.E., a noted Canadian engineer, in a design for this same crossing made some two decades ago.

knows, cover all cases of cantilever bridges having central openings in excess of five hundred feet. According to Merriman and Jacoby there were in 1907 at least forty other cantilever bridges of all kinds in America. Some of these are undoubtedly of historic interest; for instance, C. Shaler Smith's Kentucky River bridge, C. C. Schneider's Niagara River bridge, and others may have individual peculiarities worthy of mention, but the already excessive length of this chapter and the general space limit of the book prevent their receiving consideration.

The boldest bridge design ever made is undoubtedly that of Charles Evan Fowler, Esq., C. E., the well-known engineer-author, for a crossing of San Francisco Harbor. It was prepared in 1914, and has only lately been issued in booklet form. The proposed bridge starts at Telegraph Hill in San Francisco, and runs to Goat Island, which it crosses, and then passes over to Oakland. It is to provide for two steam-railway tracks, two electric-railway tracks, and two twenty-foot driveways; and the estimated cost of the enterprise is $75,000,000. The most interesting part of the project, of course, is the structure from Telegraph Hill to Goat Island, as that is the portion involving the deepest water and the most difficult foundations. There are five openings in this length, the three intermediate ones being all alike and the two end ones being the same as the others, except that one cantilever-arm in each is omitted, as the outer suspended spans of the structure are to rest on the shore piers. Each of the three main openings is covered by a 700-foot suspended span and two 650-foot cantilever-arms, making the total length thereof 2,000 feet. Each of the four towers has a length of 250 feet, which makes the total length of structure 9,700 feet. The clear height for about one half of each opening is 150 feet, the bottom chords of the cantilever-arms being curved downwards to within a few feet of extreme high water. The truss depth at the end of the cantilever-arms is 75 feet, and that at the centre of the suspended span is 135 feet, while the towers will have the great height of 450 feet from centre to centre of chords, or a total height, including the 50-foot finials, of 500 feet. To provide for lateral stability, while the clear space between the trusses of the suspended spans is 65 feet, that at the towers is 126 feet.

There will be four solitary piers for each tower, each pier being the frustum of a cone, spread in order to obtain the requisite bearing resistance. Some of these piers could be sunk by the pneumatic process, but others would have to be put down by open-dredging, the greatest depth of foundation below water level being, probably, as much as two hundred feet.

Fig. 25*ee* shows a map of the main crossing, an outline elevation, and plan of the proposed structure, and a profile of the eastern approach; and Fig. 25*ff* presents on a larger scale a side elevation and a cross-section of the structure near one of the towers. The cuts for these two drawings and that for Fig. 25*kk* were obtained through the courtesy of Mr.

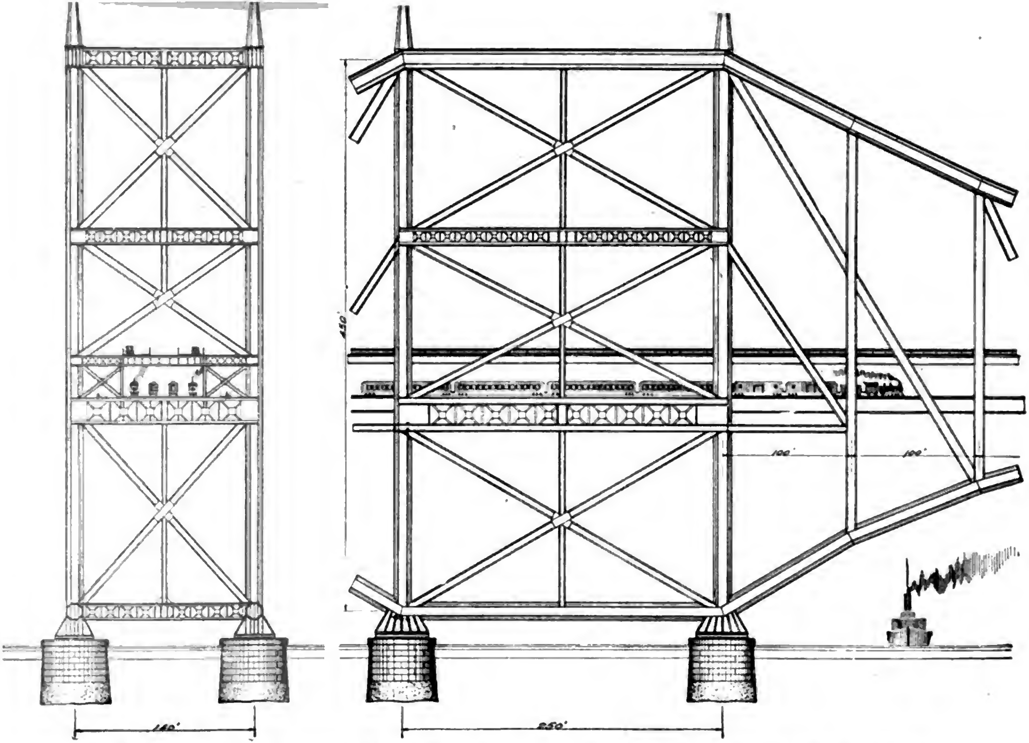

FIG. 25*ff*. Side and End Elevation of Piers and Towers of Proposed Cantilever Bridge over San Francisco Bay.

Fowler, being electrotypes of those which he used in preparing his booklet entitled "San Francisco-Oakland Cantilever Bridge." The general lines of the design are truly graceful; and there is nothing impracticable in the proposed construction, unless it be the raising of the money to build it.

Some five years ago the author was requested to make an estimate of cost for a bridge at this crossing to carry either a double-track electric railway alone, or that and a wagonway, the only datum furnished him being a Government chart. His estimates were about $9,000,000 and $12,000,000; but the greatest span lengths assumed were not more than half of those in Mr. Fowler's design, which, of course, are not the economic ones for the conditions, but are those which he must have deemed would be required by the War Department, and possibly also by the City of San Francisco. The author had an idea that spans of one thousand (1,000) feet and less would be permitted; and certainly, there is no telling what the United States Government would require until a layout is officially submitted to its engineers with a formal request for approval.

Mr. Fowler deserves great credit for his æsthetic study of the project; but, if he has any hope of ever seeing it consummated, it would be well for him to reduce his loading to one of the two adopted by the author, and let the steam railway trains enter the city by going around the Bay. The city assuredly needs the bridge for the benefit of the people of Oakland, Berkeley, and the other near-by residence towns and districts; and there is enough of that class of business today to pay for operation, maintenance, and interest on ten millions of dollars, so that the electric railway bridge would probably be a good business venture; but it is more than doubtful whether the wagon and automobile traffic would pay for the up-keep of the required roadway—and certainly it would not be great enough to provide also the interest on the additional cost of construction that it would involve.

The author's estimate may be entirely wrong; for the War Department might insist on the adoption of much longer spans than those he assumed. This question of minimum span-length is a most important one in determining the cost; and its settlement on a reasonable basis might permit some kind of a bridge to be built across the harbor in the near future. The lighter structure would not need more than four or five years to build, while Mr. Fowler estimated on ten for his heavy one.

The detailing of cantilever bridges differs essentially from that of simple span structures in only the five following connections:

1. Attachment of suspended span to cantilever-arm.
2. Transmission of wind load from suspended span to cantilever-arm.
3. Support of tower columns on main piers.
4. Attachment of anchor-arm to anchor bars and to anchor piers.
5. Attachment of anchor bars to anchorage masonry.

Each of these details will be discussed in the order named.

The ordinary method of attaching the suspended span to the cantilever-arms is by means of long, vertical hangers reaching from the pin at the end panel-point of the top chord of the cantilever-arm to the end pin of the bottom chord of the suspended span. These hangers should be rather narrow, thick eye-bars and not built members, because of the tendency to bend them in case that they fail to rotate on the pins under the effects of changes in temperature and loading. If the bearings were kept well lubricated, the hinge might be effective; but if the joint binds hard, the hanger will bend, and hence it should be proportioned to resist the combined tension and bending. As an eye-bar can bend much more readily than a rigid member, it is preferable; and as the narrower the bar the more easily it will spring, it is advisable in most cases to make the section narrow.

Some three years ago when the Dominion Bridge Company first contemplated floating into position the suspended span of the new Quebec Bridge, the author, recognizing the necessity for making the connection quickly so as to avoid as much as possible the element of danger from heavy wind, evolved a detail that would permit of the complete attachment of the span within ten minutes after its arrival at the site. He sent the said detail with his compliments to his old friend, Phelps Johnson, Esq., C.E., the president of the Dominion Bridge Company, asking merely that he be given credit for its evolution. Since then he has found occasion to adopt it for his proposed bridge over the entrance channel to Havana Harbor, but in that case the hangers are short and wide, hence the joints will have to be kept properly lubricated.

Fig. 25*gg* illustrates the detail sent to the Dominion Bridge Company. Its essential characteristic is the enlarging of one half of the pin hole, and elongating it also, in each of the two pieces that are to be connected, so that if the span be floated in at a little higher elevation than the final one, the enlarged portions of the holes in the pieces can be brought opposite each other, and the pin can be pushed through by electrical power without touching the eyes, then by letting water quickly into the barges the span can be dropped so that the pin will come to a bearing on the tightly fitting portion of each of the holes. The open space left in the other half of the pin hole can do no harm, provided, of course, that the metal there be always kept properly painted.

An effective connection of the suspended span to the cantilever-arm so as to transmit the wind load is not an easy one to design. Fig. 25*hh* shows the method that the author evolved for the before-mentioned Havana Bridge. It consists in carrying the wind load on the suspended span from the end lower lateral strut by a special strut and intermediate bracing into the end floor-beam of the suspended span, which beam is made double-webbed. These webs are slotted and properly stiffened so as to receive loosely in a vertical direction and tightly in a horizontal one a large tongue extending from the end floor-beam of the cantilever arm, which tongue is very rigidly and effectively attached to the double web of the said end floor-beam. This tongue cannot receive any vertical load, but it will take the wind load from the end floor-beam of the suspended span to the end floor-beam of the cantilever arm without causing any unduly great stress in any member or detail of the bridge, and it will

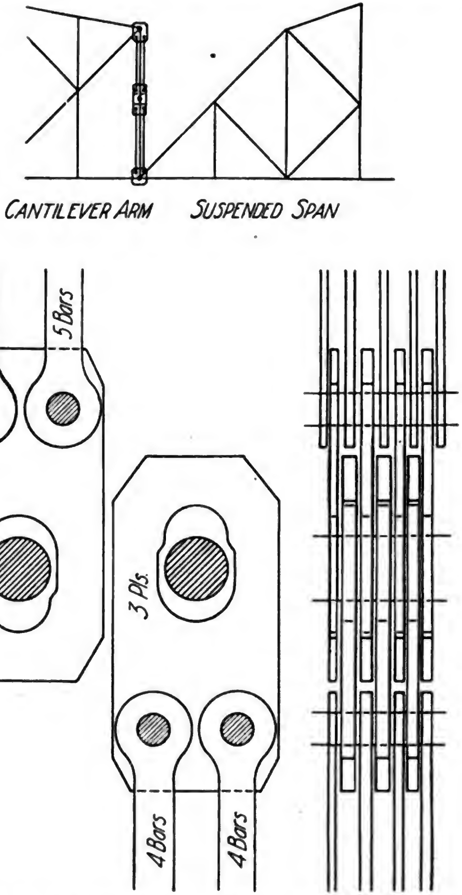

FIG. 25gg. Detail of Connection between Suspended Span and Cantilever Arm for Quick Erection.

permit of the free expansion of the suspended span. This detail will be required at the expansion end only of the suspended span; for at the other end no longitudinal movement need be provided for, and a direct connection can, therefore, be used.

In Fig. 45*a* is shown a somewhat different detail for transferring the wind load from the suspended span to the cantilever arm. It was evolved

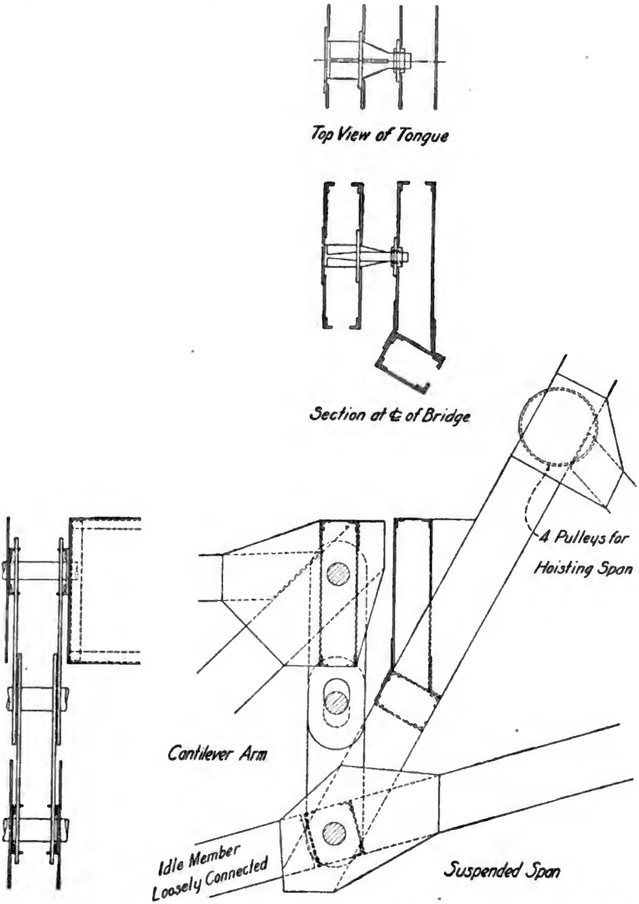

Fig. 25*hh*. Detail of Connection between Suspended Span and Cantilever Arm for the Transmission of Wind Loads.

by Ralph Modjeski, Esq., C.E., for the Thebes Bridge over the Mississippi River.

In most cantilever bridges the columns over the main piers rest on ordinary pedestals, which generally are not lubricated; hence it is not at all sure that the hinge acts effectively—in fact it is pretty certain that it does not. Fig. 25*ii* shows a most efficient detail for this connection, prepared by Albert Lucius, Esq., consulting engineer of New York City, for the Beaver Bridge, which structure is described previously in this chapter. Its essential feature is a nest of segmental rollers in a cylindrical shaped bearing, each roller having bolted to it at each end a plate with toothed ends that fit into a rack on top and into the base casting below. Such detailing as this is a good indication of the progress that has been made of late years in bridge designing in the United States, and is a proof that bridge building can now very properly be termed a science.

In the Beaver Bridge there was perfected also the method of attaching the anchor-arm to the anchor bars and to the anchor piers. This anchorage (see Fig. 25*jj*) takes care of a reversal of stress due to the moving load. The device consists of a heavy steel casting arranged to carry three wedges which support a frame through which the connecting pin passes. This pin is anchored to the masonry by eight heavy eyebars and supports two standards or rockers which extend to the end pin in the lower chord. This permits of a horizontal motion of the anchor-arm, due to temperature variations, without putting bending on the anchor bars. These bars were put under an initial tension by loading the cantilever-arm and the suspended span with a special live load in excess of the working live load. Then the wedges before mentioned were forced tight to their bearings and secured in place. The horizontal component of the stress in the rockers is taken up at the lower pin by means of jaws and a slide in the pedestal casting; and it is thus transferred directly to the masonry.

The attachment of anchor bars to the anchorage masonry is an important matter and requires careful detailing. The stress in the eyebars must be transferred to the masonry in such a manner that the unit bearing capacity of the concrete will not be exceeded. This detail was thoroughly worked out in case of the aforesaid Beaver Bridge. In this case the load on the anchor bars was transferred to a pin which was carried by an inverted shoe having a large base area and bearing against the lower side of an I beam grillage embedded in the masonry of the pier. This grillage has sufficient bearing area against the masonry so that the unit load is within the safe limit.

As a last word on the special detailing of cantilever bridges, there is herewith reproduced in Fig. 25*kk* a typical cross-section of a compression chord member in Mr. Fowler's design for the San Francisco Harbor Bridge, together with the make-up of the sectional area thereof. This octagonal section is stiffer than the ordinary rectangular one and

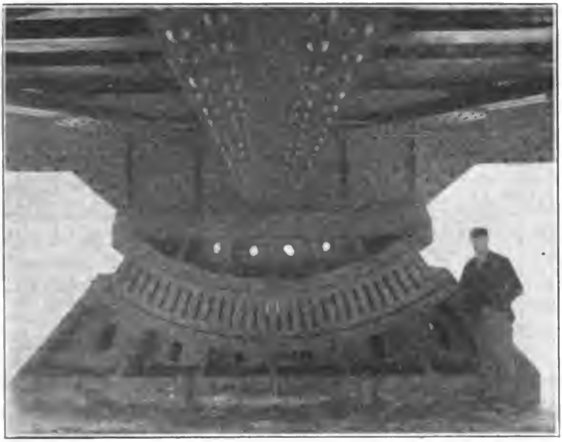

FIG. 25*ii*. Main Shoe of the Beaver Bridge.

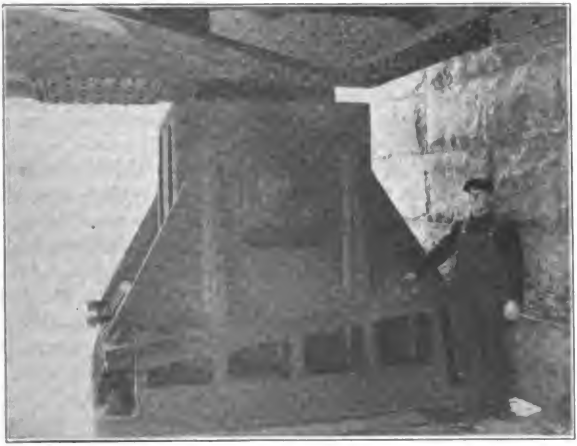

FIG. 25*jj*. Anchorage for the Beaver Bridge.

much better for connecting to than the circular section employed in the chords of the great Forth Bridge. One of its special advantages is that it can be fabricated in sections in the shop and built up into final shape in the field. It affords ample room for connecting the web members by either gusset plates or pins. The various diaphragms employed stiffen the section so that it is even stronger in its individual parts than as whole. The bending due to the great unsupported length is taken care of by the extra plates at the top and the bottom of the section. Painting on the inside is cared for by leaving man-holes in the diaphragms so that all parts of the metal can be reached by the paint brush.

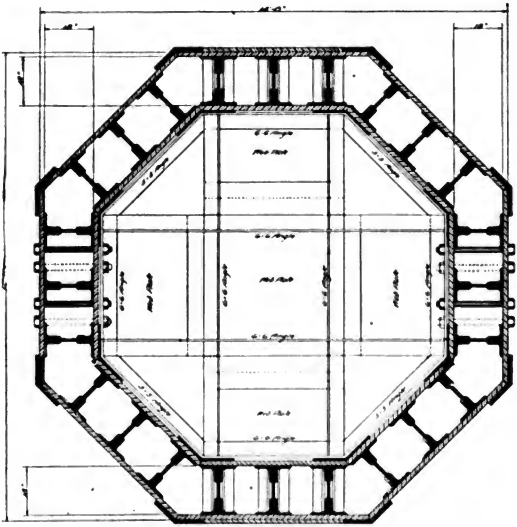

FIG. 25*kk*. Typical Cross-section of Compression Chord for Proposed Cantilever Bridge over San Francisco Bay.

CHAPTER XXVI

ARCH BRIDGES

As pointed out in Chapter I, the masonry arch is one of the oldest types of extant bridges. Its use has continued uninterruptedly throughout the ages until the present time, but it is now rapidly being replaced by the reinforced concrete arch, with the imminent prospect that ere long it will not be employed any more. On this account and because the theory of its stress distribution and the details of its construction have been adequately treated in a number of standard works, such as those of Howe, Baker, Cain, and Burr in America, and Séjourné and others in France, no attention will be paid to it in this chapter, excepting only to remark that the theory which locates the centre line of pressures is based upon assumptions which cannot be verified, but which have been proved by long experience to be sufficiently accurate for all practical purposes. The elastic theory is applicable to the masonry arch as long as there is no tension at any joint. For its derivation the reader is referred to "Modern Framed Structures," Part II. Prof. Wm. H. Burr, the well-known engineer-author, in the *Selected Papers of the Rensselaer Society of Engineers*, some thirty-five years ago, gave a very satisfactory demonstration of the method of locating the curve of pressures in the masonry arch, and Professors Cain and Howe in their well-known books on arches treat the subject thoroughly. Nor will the reinforced concrete arch be discussed at present, as its treatment will be covered in Chapter XXXVII. To devote any attention at all to the subject of timber arches would involve a total waste of both book-space and readers' time; for such constructions have become so antiquated, and the conditions conducive to their adoption so rare, that no mistake will be made in terming them obsolete. There remain, then, only metal arches with which to deal; and these reduce to steel ones, as nobody to-day would think of adopting therefor either cast or wrought iron.

Arches are employed very generally in Europe on account of their superior appearance as compared with simple-truss bridges, and because of the powerful influence of the old masonry arch upon the minds of European bridge designers, regardless of the consideration of economy. American engineers, on the other hand, have been indifferent to the question of æsthetics, and have preferred simple spans to arches mainly for reasons of simplicity and economy, but sometimes on account of their greater rigidity. Another reason why the arch has not been used much in American practice is that the conditions which make it economical are not met with as frequently in this country as in Europe. For deep gorges with rocky sides, or for shallow streams with rock bottom and natural abutments, arches are eminently proper and economical. But when a steel bottom chord is needed to take up the thrust between springing points, all the economy of the arch vanishes.

The advantages of the arch are a possible economy in cost of metal and an æsthetic appearance, while its disadvantages are a lack of rigidity and, for most types, an uncertainty concerning the stresses in the members.

Arches are sometimes used for large train-sheds, in which their architectural effect is certainly very fine, but they require about twice as much metal as do cantilevered trusses supported on columns; consequently they should be adopted only when appearance is an extremely important factor in the design.

When bridge foundations have to be built on piles or on any other material that is liable to slight settlement, or when the abutments could possibly move laterally even a mere trifle, it is not proper to adopt an arch superstructure; for any settlement or any motion whatsoever in either piers or abutments would upset the conditions assumed for the computations, and thus cause to be increased to an uncertain amount some of the stresses for which the superstructure was proportioned. This criticism does not apply to the three-hinged arch, but even that type requires good, solid abutments and firm foundations for piers; because, while a vertical settlement of the supports would do no harm, a horizontal displacement thereof would cause a lowering of the crown, which, if unchecked, would overstress the metal and eventually destroy the structure.

Arches can be erected on falsework, by cantilevering, or by building vertically the two halves and lowering them by cables till they meet at the centre. Whichever of these methods is the easiest and cheapest is the one to adopt. A very easily erected arch is shown in Fig. 26*a*. The pieces marked *AB* are temporary, and are to be used only during erection. They can be made of timber, so as to be removed readily after the arch is coupled at mid-span, or they may be of steel and be left in as idle members, solely for the sake of appearance. It will be seen from the diagram that the structure is a cantilever during erection, and afterward consists of an arch span and two simple spans. This type of bridge probably requires a little more metal than would an ordinary arch with trestle-approaches, and possibly is not quite as rigid as the latter; but the saving of cost in erection will generally much more than offset these disadvantages. The fact that the arch type of structure lends itself readily to erection by cantilevering is one of the strongest reasons for its employment in bridge building; because in mountainous countries there are many gorges to cross where erection by means of falsework would necessitate excessive expense.

There are four cases all told, so far as the number of hinges is concerned, viz.:

1. Arch without any hinges.
2. Arch with one hinge (at crown).
3. Arch with two hinges (at abutments).
4. Arch with three hinges (at crown and abutments).

In Case No. 4 there are no temperature stresses nor indeterminate stresses of any importance, but in all of the other cases there are; and they must always receive due consideration in proportioning the members.

All things considered, the author prefers to adopt the three-hinged arch for railroad bridges, because the stresses can be determined as accurately as can those of an ordinary truss bridge, and because of the absence of temperature stresses; but at the same time it must be admitted that an arch without hinges is more rigid than one with hinges, and that, theoretically, it is claimed by some engineers to be more economical of metal.

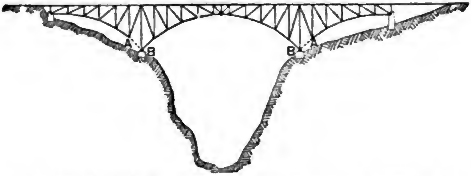

Fig. 26*a*. Layout of Arch Span with Arch-like, Simple-truss, Flanking Spans.

In the *Engineering Record*, Vol. 68, page 321, there is a valuable article by the well-known American bridge engineer, Ralph Modjeski, Esq., treating of steel arches. He disagrees with the author concerning the best type of arch for railroad bridges, preferring two hinges to three. However, he advocates designing such structures so as to have three hinges for the dead-load stresses and only two for the live-load stresses; and this, in ordinary cases, does away with about one-half of the objectionable ambiguity of stress distribution in the two-hinged arch. It is accomplished by putting in a hinge at the centre, preferably at mid-truss depth, erecting the metalwork and the floor so as to bring the full dead load on to the arches, then making the top and bottom chords continuous by riveting in special sections thereof designed to carry the greatest possible stresses from live load and impact—and in some cases also from wind pressure. The resulting construction is certainly satisfactory. For highway bridges, in which the assumed live loads will seldom, if ever, be realized, it might sometimes be best, all things considered, to adopt the arch without hinges, so as to obtain the greatest possible rigidity, even at the expense of certainty in computing stresses. For arched train sheds, the two-hinged arch of crescent shape will generally be found the most satisfactory.

The framing for arches is ordinarily one of the three following types:

1. Solid-rib.
2. Braced-rib.
3. Spandrel-braced.

The solid-rib type consists of a curved plate-girder rib, carrying the roadway on posts resting on the top or by suspenders hung from the bottom—usually the former. This type is illustrated in Fig. 26*i*.

The braced-rib type consists of two parallel, or nearly parallel, chords at some distance apart, connected by a system of open webbing, the roadway being carried in either of the ways just mentioned. Fig. 26*f* illustrates this type with posts supported on the arch.

The spandrel-braced arch is applicable only to deck structures. It consists of a curved bottom member, which is the main arch rib, a horizontal top chord near the plane of the floor, and web trussing, usually of the Pratt type. (See Fig. 26*a*.) A special case of the spandrel-braced arch is the cantilever arch, an example of which is shown in Fig. 26*l*.

The trussing of the webs of the ribs should invariably be of single cancellation; for there is, in all conscience, already too much ambiguity in arch designing to warrant still further complication by the adoption of a multiple-intersection system.

In respect to the question of the comparative merits of half-arches of the lenticular and the parallel curve types, as shown in Figs. 26*b* and 26*c*, the author once had occasion to investigate the economics of the matter for a 260-foot span, and found that the weights of metal required were almost exactly the same; hence he adopted the parallel curve type for æsthetic reasons.

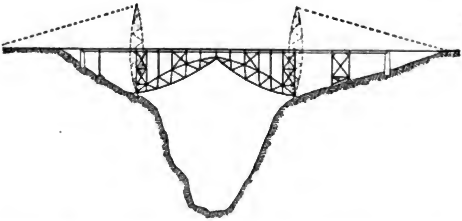

FIG. 26*b*. Three-hinged, Lenticular, Braced-rib Arch.

Although arches without hinges are quite common in Europe, there are very few of them in America, the most noted ones of this type being the Oakland Bridge with its single span of 440 feet at Pittsburgh, and the Eads Bridge at St. Louis with its three spans, one of 520 feet and two of 502 feet each. The longest hingeless arch span in the world is in the Kaiser Wilhelm Bridge at Mungsten, Prussia, its length being 557 feet between tower centres. This type of arch almost invariably is of pleasing appearance, as any one who has had the opportunity to view the Eads Bridge will readily testify. Its objectionable features are the unavoidable ambiguity of stress distribution, the high temperature stresses, the immense amount of labor involved in making the computations, the heavy detailing required for truly fixing the ends by attaching them to the masonry of the piers and abutments, and the baneful effects of even the slightest motion of the abutments either vertically or horizontally, especially in flat arches. It is claimed that hingeless arches require less metal than any other type; but the author, upon general principles only, is inclined to disagree with that opinion. He has never had occasion to design an arch without hinges, but feels quite confident that if he should ever figure one, he would find that a proper provision for excessive temperature stresses, possible large indeterminate stresses, unusual detailing, heavy anchor-bolts, and an inherent sense of fitness in proportioning sections, would so increase the weight of metal as to absorb all of the theoretical economy over the three-hinged arch involved by the smaller average direct stresses as shown on the stress sheets. Were it not for the immense amount of labor required and for the lack of time at his disposal for the finishing of the manuscript of this book, the author would settle this question finally by making several complete detailed designs and computing the resulting total weights of metal. A correct comparison could be obtained in no other way—moreover, it would have to be made by an expert bridge designer, who would know how to care properly for all the unusual stresses and conditions. No short cuts to results could be employed, such, for instance, as multiplying each stress by the length of the piece and summing the results.

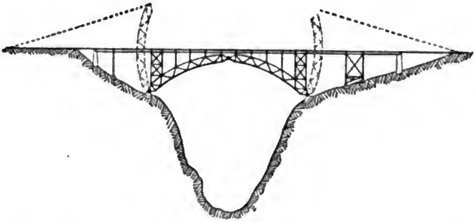

FIG. 26c. Three-hinged, Braced-rib Arch with Parallel Chords.

Hingeless arches may be constructed of either the solid-rib or the braced-rib type; but the spandrel-braced type does not lend itself readily to that class of structure, because the ends would nearly always be hard to fix and because it would be very difficult to make the preliminary approximations of sections with sufficient exactness at the first, the second, or possibly even the third trial, thus necessitating an almost endless amount of computation to prepare the final design. In "Modern Framed Structures," Part II, will be found the mathematical theory of the hingeless arch, covering both the solid-rib and the braced-rib types. For economy the rib should be shallow at the centre and deeper at the ends, and this provision produces the most artistic appearance attainable. It can be erected as a one-, two-, or three-hinged arch, and the joints can afterward be riveted up at some specified temperature, thus reducing somewhat the stress uncertainty, especially as the effects of fabrication and construction errors can then be eliminated. This carrying of the dead load as a hinged structure theoretically increases the amount of metal required, but practically the smaller amount needed to provide for uncertainty of stresses will about offset the excess.

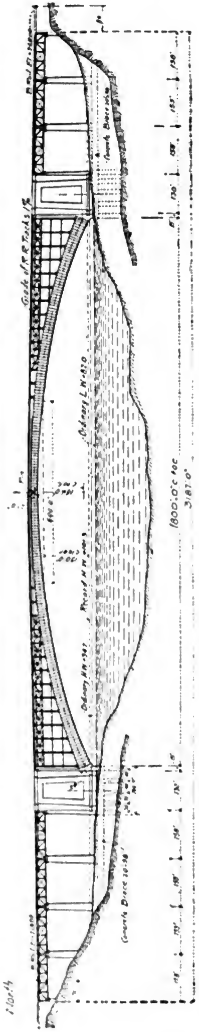

FIG. 26*d*. Proposed Arch Bridge over the St. Lawrence River at Quebec.

Until lately the one-hinged arch has been unknown in America, although it has been recognized as a possibility; but in the May 19, 1910, number of *Engineering News* there appeared a design by Charles Worthington, Esq., M. Am. Soc. C. E., for a one-hinged steel voussoir arch of 1,800 feet span to replace the Quebec Bridge that failed. This is one of the boldest bridge designs ever presented to the public; and, moreover, unlike some of its predecessors in novel constructions of great magnitude, it appears to be quite feasible. The designer's presentation of his project is simple and clear, but, of course, his estimates would require careful checking before acceptance. Fig. 26*d* gives a very good idea of the general appearance of the proposed structure.

The principal novelties of the design are as follows:

First. The unprecedentedly great span length for arch construction; for it is nearly twice as long as the longest arch yet built.

Second. The use of steel voussoirs like those of stone, and the non-reliance on the ribs to resist bending. This idea, however, in a way is not new; for, as stated in Chapter I, cast-iron voussoir arches were built in Europe before wrought-iron bridges came into vogue, and formed the connecting link between the masonry bridges of old and the metal bridges of the present time. Mr. Worthington's proposed voussoirs are immense, being nine (9) feet long, nine (9) feet wide, and varying from twenty-one (21) feet deep at the crown to forty-three (43) feet deep at the skew-backs. He estimated upon making them of nickel steel, and stressing the metal up to 28,000 pounds per square inch in compression, which is about the intensity that the author, in his paper on "Nickel Steel for Bridges," shows to be correct.

Third. The assumption of a certain curve to suit the requirements for underneath clearance, and then forcing the centre lines of pressure for dead load only to follow exactly the centre lines of the voussoirs. This was to be accomplished by loading the four arches with concrete wherever it might prove necessary. Of course, such loading would be a bit of extravagance; because it would be much better to adopt such a curve that the line of pressure for each arch would follow the line of symmetry without forcing, and then induce the Dominion Government to approve the lines of the layout. The idea, though, is a clever one, and, as such, is worthy of commendation.

Fourth. The method of erecting by building a temporary suspension bridge of steel cables and lowering from it the voussoirs is novel in a way, although the erection of truss bridges by temporary suspension structures is an old scheme. The magnitude of the method for this case is somewhat overwhelming, for the cost of erection would be immense; but, on the other hand, the salvage would be proportionately large, because the ropes would be very little, if at all, injured by their service.

Fifth. The buttressing of the abutments by immense concrete slabs carried from their rear below the ground to the rocky sides of the ravine is unique and perfectly practicable. Moreover, it ought to be effective; for the expansion and contraction of deeply buried concrete from changes in temperature should be exceedingly small.

Sixth. The adoption of a hinge at the crown ensures that at that point the centre of pressure will, for all loadings, lie upon the line of symmetry, which is not the case in masonry-arch construction, except in a few instances where hinged castings with pins have been employed.

Mr. Worthington's design, as can be seen from the illustration, is certainly æsthetic; and in this particular it stands out in vivid contrast to the rather inartistic cantilever design evolved by the Commission of Engineers. It was not accepted by the latter; for, as was stated in the preceding chapter, a cantilever bridge of the *K* system of cancellation was adopted. Whether Mr. Worthington's type of design is ever put into execution, or whether it is economic or otherwise in comparison with structures of other types, he is certainly to be commended for the boldness and beauty of his conception and for the clear manner in which he has presented it to the engineering profession. Such a design is truly a credit to American bridge engineering; and it should be a source of satisfaction to the members of that specialty in this country, in that it affords a basis of their claim that American bridges in general are inferior in appearance to those of Europe mainly because of financial restrictions and not on account of any deficiency in artistic taste among American engineers.

The one-hinged arch is fairly rigid, the stresses in it are more easily calculated than those in the hingeless arch, the results of the computations are more certain, the temperature effects are less, and slight movements in the supports would not cause such excessive increases in the stresses. The solid-rib and the braced-rib types are applicable to this type but not the spandrel-braced rib.

The two-hinged arch, like the arch without hinges, is quite common in Europe but rather unusual in America. It is not as rigid as those that have fewer hinges, and its temperature stresses run high, especially in flat arches, but by no means as high as those in the hingeless type. The calculation of the stresses is rather long and tedious, and there is some uncertainty involved in their solution; but a complete analysis thereof is much simpler than in the case of the arch without hinges. The stresses are increased considerably by any lateral movement of the supports, the tendency of any such motion being to make the arch act like a simple truss. It can be designed of the solid-rib, braced-rib, or spandrel-braced type. When either the solid rib or braced rib is employed, the theoretic outline would require the depth to be greatest at the haunches, decreasing somewhat at the centre and reducing to zero at the ends; but such a form would be unsightly. The crescent shape has sometimes been adopted, as in the Garabit Viaduct in France. That arch has a span

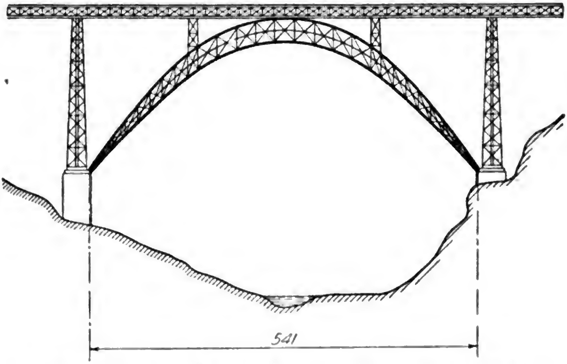

FIG. 26c. Garabit Viaduct over the Truyère River in France.

of 541 feet and a rise in the lower chord of 166 feet, the truss depth at the centre being about 33 feet. (See Fig. 26*e*.)

A better and more usual method is to make the ribs of constant depth throughout, as in the case of the 840 foot span of the Niagara and Clifton Bridge, shown in Fig. 26*f*, or in that of the 510-foot span, solid-rib arch of the Washington Bridge at New York City. The end panel in each of these cases tapers down to a point at the pin. In other cases the ribs have been made deeper near the ends, as in the case of the 977-foot span of the Hell Gate Bridge now under construction, shown in Fig. 26*g*. As this, when completed, will be the longest arch span in the world, it is worthy of a full description. Its designer, the noted bridge engineer, Gustav Lindenthal, Esq., was kind enough to furnish the author with the following data and pictured layout for the bridge and its approaches:

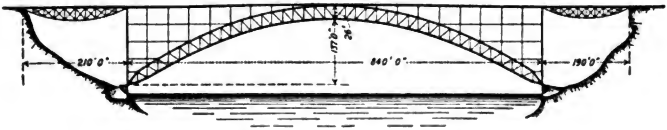

FIG. 26*f*. Niagara-Clifton Bridge over the Niagara River.

HELL GATE ARCH BRIDGE

Length, 1016′ 10″ between tower faces,
995′ 1⅜″ from centre to centre of bearings on masonry,
977′ 6″ from centre to centre of end hinges.

Rise of intrados, 220 feet.

Height of top chord of arch above elevation of end hinges at end posts, 140 feet; and at centre, 260 feet.

Width, 60 feet from centre to centre of trusses; 93 feet from centre to centre of railings.

Maximum cross-sectional area of bottom chords, 1,385 square inches.

Heaviest bottom chord section, 150 tons (shipping weight).

Total weight of steel (high carbon), 20,000 tons.

Total dead load per lineal foot, 50,700 pounds.

Live load, 4 tracks, Cooper's E-60 loading.

ARCH BRIDGE TOWERS

Reinforced concrete structure with granite facing over all, heavily reinforced track floor, and structure architecturally treated with granite above track level.

WARD'S ISLAND TOWER

Founded on 21 pneumatic concrete caissons carried down to rock (15 cylindrical, 18 feet diameter, and 6 rectangular, 30 ft. × 41 ft., keyed together).

Maximum depth of single caisson, 109 feet below mean low water.

Total foundation masonry, 28,000 cu. yds.

Masonry above foundation (concrete and granite), 38,000 cu. yds.

LONG ISLAND TOWER

Founded on rock (encountered about 15 ft. below ground surface).
Total foundation masonry, 5,000 cu. yds.
Masonry above foundation (concrete and granite), 37,300 cu. yds.
Dimensions of towers at ground surface, 140 ft. × 104 ft.
Top of tower, 250 ft. above mean low water.

Track floor, 150 ft. above mean low water.
Under clearance of steel structure, 140 ft. above mean low water.
Largest gussets, (1) 120″ × $1\frac{5}{8}$″ × 17′ 6″.
(6) 126″ × $1\frac{5}{8}$″ × 14′ 6″.
Longest rivets (field), $1\frac{1}{4}$″ diameter and $9\frac{7}{8}$″ grip.

The picture shows only a part of the viaduct on each side of the arch span. In reality the bridge and its approaches are over 18,000 feet in length, the latter consisting of concrete arches, plate-girder viaducts, bascule spans (over the Bronxkill), and an inverted bow-string bridge about 1,000 feet long over Little Hell Gate, the whole work requiring about 90,000 tons of steel and about 460,000 cubic yards of masonry.

The bridge itself is certainly of æsthetic appearance, and it reflects great credit upon the artistic ability of its designer. The deep trusses at the ends were evidently adopted for the sake of appearance; and it would be interesting to know how much metal and money could have been saved by making the arch of crescent form.

Other two-hinged arches of long span are to be found in the Bonn Bridge across the Rhine in Germany, which structure has a span of 614 feet, in the Düsseldorf Bridge over the same river, with its two arched openings of 595 feet each, and the Grand Trunk Railway Bridge over the Niagara Gorge with its 550-foot span. The latter is an example of the two-hinged, spandrel-braced arch.

In two-hinged arches the lower chord is usually either circular or parabolic, the latter form being the more economic. The calculations are made by the general method of deflections given in the standard treatises on bridge stresses. As before mentioned, two-hinged arches can be erected as three-hinged ones for the dead-load stresses and then made continuous at the crown for the live-load stresses. In such cases, for spandrel-braced arches there is an increase in weight if the temporary hinge be placed in the bottom chord and a slight saving if it be placed in the top chord. This addition of a temporary hinge is generally desirable.

The three-hinged type of arch is the least rigid of all the types, but is otherwise the most satisfactory; for the temperature stresses are practically nil, being confined to those caused by the decrease in the effective rise of the arch due to falling temperature. The stresses caused by slight lateral movements of the supports are of the same nature and small, and there is no ambiguity of stress distribution whatsoever anywhere in the structure. The calculations of stresses are simple and, comparatively speaking, not lengthy, the best method therefor, when the arches are open-webbed, being to place a unit load at each panel-point and to find by graphics the stresses caused by it in all the truss members, taking care to note for each stress whether it is tension or compression, then to record all the stresses from the various unit loads in a table. By using a slide rule the stresses due to the actual loads can easily be found, then tabulated and summed, care being taken in the summation to ignore all impossible or extremely unlikely combinations of live loads. It may be necessary to use different panel dead loads at the various panel-points, especially in long-span bridges. The unit load method of figuring the stresses is specially applicable to this condition of unequal loading.

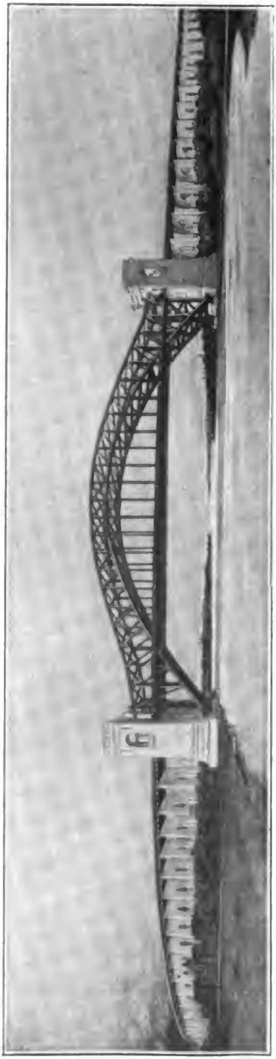
FIG. 26*g*. Hell Gate Arch over the East River, New York City.

Three-hinged arches can be of the solid-rib, the braced-rib, or the spandrel-braced type. Where the two first-mentioned types are employed, the theoretic outline is narrow at the crown and springing points and deeper at the haunches. Such an outline has been used occasionally, as in the 460-foot span of the Austerlitz Bridge over the Seine in Paris, in the 363-foot span of the Alexander III. Bridge at the same location, and in a few structures of novel form such as the Assopos Viaduct in Greece. The Austerlitz Bridge, Fig. 26*h*, should be noted particularly, because in it the end hinges are placed at some distance from the skew backs, the ribs being fixed to the abutments. The increased haunch thickness is not graceful; and it has been more usual to make the ribs of constant depth as in the plate-girder ribs of the 340-foot span of the author's bridge over the Waikato River at Hamilton, New

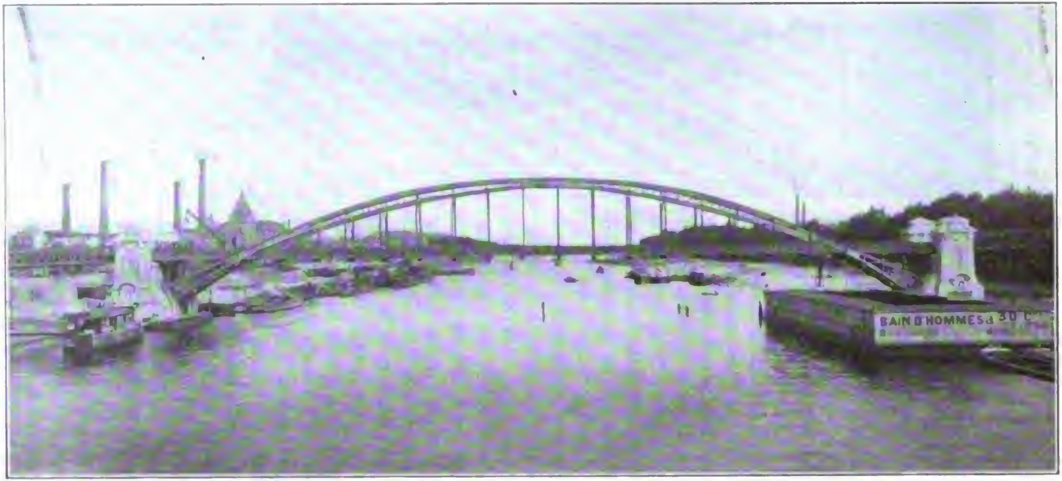

FIG. 26*h*. Austerlitz Bridge over the Seine, Paris.

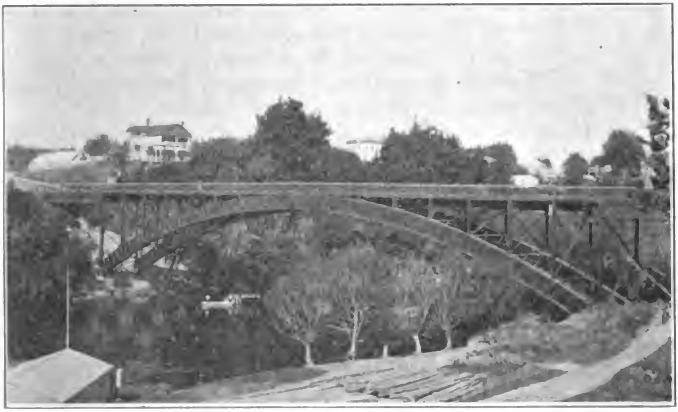

FIG. 26*i*. Arch Bridge over the Waikato River at Hamilton, N. Z.

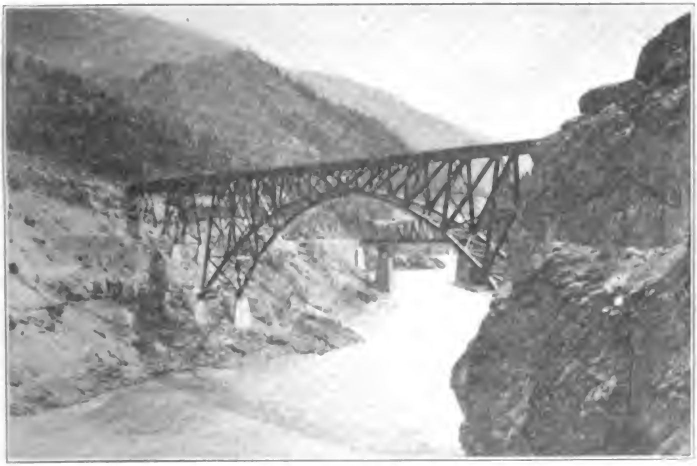

FIG. 26*j*. Canadian Northern Pacific Railway Bridge over the Fraser River at Lytton, B. C.

Zealand, shown in Fig. 26*i*, and in the 540-foot, open-webbed span of the Bellows Falls Bridge over the Connecticut River. In most of the bridges of this class the end panels of each rib adjacent to the pins bevel down to meet the pin theoretically at a point; but in the Bellows Falls arch this construction is employed at the ends only, making the depth of truss elsewhere constant, thus adding to the appearance of the structure. The pin at the crown is carried by special web members.

The three-hinged, spandrel-braced arch is similar in appearance to the two-hinged. The hinge is usually placed in the bottom chord, as in the 456-foot, wrought-iron spans of the Lake Street Bridge over the Mississippi River at Minneapolis, in the author's 425-foot steel span over the Fraser River on the line of the Canadian Northern Pacific Railway in British Columbia, shown in Fig. 26*j*, and in the author's 290-foot span of the Cambridge Bridge over the Waikato River in New Zealand, illustrated in Fig. 26*k*. According to Merriman, however, there is a saving of metal effected by raising the crown hinge. He analyzed the two-hinged, spandrel-braced Niagara arch and found a saving of main sections amounting to 0.8 per cent by the use of a crown hinge in the lower chord, one of 8.8 per cent by placing it midway between the chords, and one of 11.8 per cent by putting it in the top chord. This comparison does not include the weight and extra cost of the crown hinge, which in all

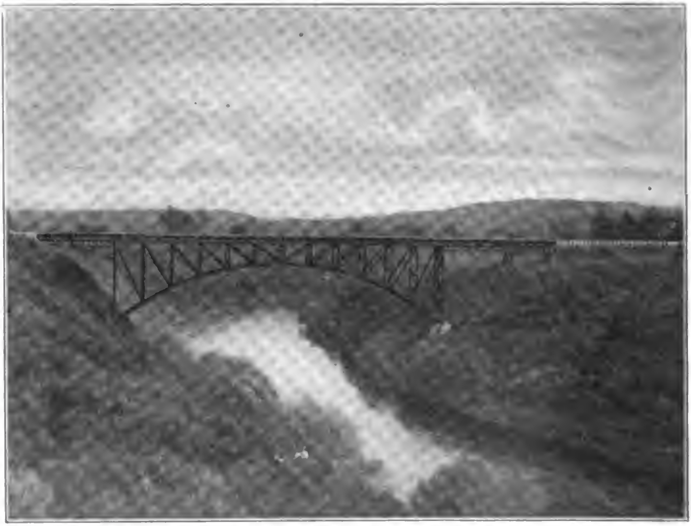

FIG. 26*k*. Arch Bridge over the Waikato River at Cambridge, N. Z.

probability will overcome the saving in the first case and reduce it somewhat in the last two. These results appear surprising to the author, not only because of the great variation involved by locating the pin at different places, but also because the general opinion of bridge engineers is that, *theoretically at least*, the two-hinged arch requires on the average somewhat smaller sectional areas of truss members than the three-hinged arch.

In the design of the 225-foot, spandrel-braced arch over the Menominee River in Michigan, a parabolic lower chord was first tried with the crown pin on its centre line; then later there was worked out a design with hyperbolic bottom chord and the hinge half way between the chords. This partly avoided the reversing stresses in the spandrel members and secured a much lighter structure. A third design with hyperbolic lower chord and pin located thereon was then tried, the resulting weight of metal proving to be intermediate between the other two. The hyperbolic curve with crown hinge midway between chords was finally adopted.

The saving in weight by raising the crown pin is probably due to the following causes:

1. The horizontal thrust is lessened and, consequently, its effect on the arch members throughout is reduced.

2. There is an avoidance of some reversion of stresses.

3. With the pin on the bottom chord there are certain members of the top chord and of the web near mid-span which have excessive section; and the raising of the hinge brings this idle metal into play and at the same time reduces the stresses in the bottom chords for several panel lengths on each side of the centre.

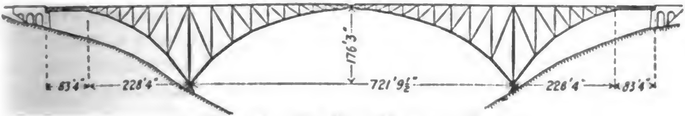

Fig. 26*l*. The Viaur Viaduct in France.

The author's reason for placing the crown hinge on the bottom chord was that it appeared to him logical and in line with æsthetics, but he is now convinced that he was wrong in so doing. However, the only disadvantage that his spandrel-braced arch bridges possess is a lack of economy that is far from being excessive. In his plate-girder arches the crown pin was always placed at mid-depth of web. In the 721-foot span of the Viaur Viaduct in France, shown in Fig. 26*l*, it was located at the same position.

There is now being built at Cleveland, Ohio, over the Cuyahoga River, what is known as the Detroit-Superior Bridge, so-called because it is to connect Detroit and Superior avenues. This structure, on which the author's firm was retained as consulting or, more strictly speaking, advisory engineers, contains a three-hinged arch span of 591 feet, having lines somewhat like those of the great Hell Gate Arch, as can be seen by the layout shown in Fig. 26*m*. Its most notable features, in addition to the general truss-outline, are the details of important members, the unusual truss and floor connections, the double-deck arrangement with four cantilevered sidewalks, the wide driveways with their six electric car lines, the heavy loads assumed and the various combinations of them provided for, and the joint use of carbon and nickel steels. The main trusses are about 49 feet apart, and their depths vary from 20 feet at the crown to 91 feet at the ends. The rise of the bottom chords is 144 feet, and the vertical clearance above low water is 96 feet.

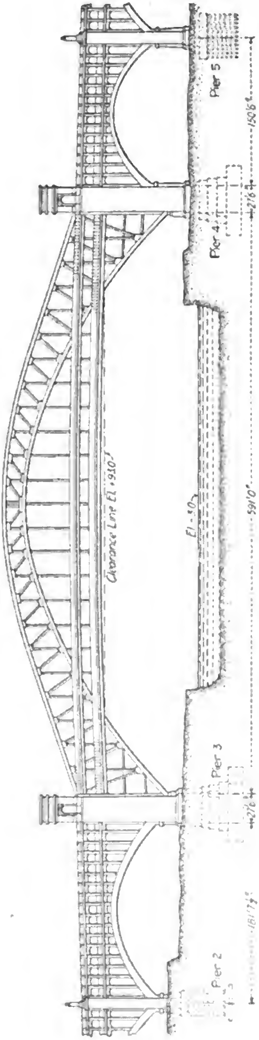

FIG. 26*m*. Detroit-Superior Bridge at Cleveland, Ohio.

The special type of spandrel-braced arch known as the cantilever arch has already been mentioned. It consists usually of a central span, which is a spandrel-braced arch, and two shorter end spans, each of which is apparently a half arch of the same type, but in reality consists of a cantilever arm adjacent to the arch and a suspended span resting on the cantilever arm at one end and on the abutment or pier at the other. Two well-known examples of this type of construction are the Viaur Viaduct, above mentioned, and the Rio Grande Bridge of the Pacific Railway of Costa Rica. The Viaur Viaduct has a central, three-hinged, spandrel-braced arch of 721 feet span, and two end spans each 311 feet long, each cantilever arm having a length of 228 feet and each suspended span a length of 83 feet. The Rio Grande Bridge has a central, two-hinged, spandrel-braced arch of 449 feet span, and two end spans of 118 feet each, the lengths of the cantilever arms being 47 feet and those of the suspended spans 71 feet.

The cantilever arch is well adapted

to certain localities. It reduces considerably the horizontal thrust on the arch pier, and probably lessens somewhat the weight of the arch span. The total vertical movement of the central point of the latter is somewhat greater than would be that of the corresponding ordinary arch of the three-hinged or the two-hinged type; but the deflection under any live load on the central span is only a little greater. It is well adapted to erection by the cantilever method, the cantilever arm and the suspended span being connected to one continuous truss during this operation. Fig. 26*l* illustrates clearly the principle of the cantilever arch bridge.

Concerning the relations between the principal dimensions for arch bridges of various types but little can be said, for the reason that but little is known. In most cases the length of span and the rise are determined by the existing conditions at the crossing. For any given span, the greater the rise the less the effect of uniform load stresses; but such a variation has little or no effect on the partial load stresses. Again, for any given span and rise, the arch-rib depth does not affect the uniform load stresses materially, while it does so affect the partial load stresses; and as the maximum moments from the latter are much less than the moments in a simple span, it results that the depth of an arch-rib for economy of material will be very much less than the best depth for an ordinary truss of the same span.

Hard and fast rules for the minimum spacing of outer arches of bridges for various spans and rises cannot well be given. The narrower the structure, within reasonable limits, the less the cost, but the less also the rigidity and the lateral resistance to overturning from wind-pressure. In the 260-foot span mentioned on page 620, the author made the distance between central planes of arches twenty-two feet, which was as small a distance as he dared to adopt, notwithstanding the fact that economy of first cost was an important factor in the design. An approximate rule to work by might be to make the perpendicular distance between outer arches at springing points not less than one third of the height from springing point to grade.

For an arch structure in which it is either difficult or impossible to secure a sufficient rise and still keep the crown below the roadway, one of the rib types, preferably the braced rib, should be adopted, and the centre portion of the rib should be allowed to rise as high above the roadway as is necessary for vertical clearance and overhead bracing. In the United States the deck arch bridge is the one commonly employed, as the arch there has been adopted almost always for the crossing of deep gorges; but the use of the suspended floor is very common in Europe, where the arch is frequently utilized for ordinary types of crossings.

The application of the arch to very long spans has occasionally been advocated with considerable plausibility, but has met with little response. Mr. Worthington's design for a span of 1,800 feet has already been described and discussed. Mr. C. R. Grimm, in a paper presented to the

American Society of Civil Engineers and printed in its *Transactions* for 1911, made a plea for the use of long-span arches of crescent-shaped ribs, claiming a decided advantage in both appearance and economy for them over the cantilever and suspension types of construction; but many of the engineers who discussed the paper disagreed with his views, deeming the other types to be preferable, or at least that Mr. Grimm had failed to present convincing proof of the correctness of his claims. Mr. R. S. Buck, who is well known in the United States and Canada as a bridge engineer, said that an arch of the rib type, such as that in the Niagara and Clifton Bridge, is subjected occasionally to very severe vibrations under live load, and that Mr. Grimm's crescent-shaped ribs would be still worse in this respect. Mr. Buck prefers the spandrel-braced arch for such structures.

The stress calculations for arches have been referred to previously. Any complete presentation of them is beyond the scope of this treatise. There are a number of books which give very good analyses, but Part IV of Merriman and Jacoby's book and Part II of "Modern Framed Structures" have been the texts usually followed in the author's office, the treatment offered in the latter being generally preferred, particularly for two-hinged and three-hinged arches. The analysis is given in that book completely for both the solid and the braced-rib types, and some illustrative examples are worked out. For the analysis of the two-hinged arch, a complete mastery of the principles explained therein on pages 148–150, Arts. 135 and 136, is advisable, as their employment will shorten materially the labor of calculating the stresses. The same points are brought out on pages 255–269 of Part IV of Merriman and Jacoby's book, although in a slightly different form. The device of summing the results for two symmetrical panels was evolved by the noted bridge expert, Theodore Cooper, Esq., when making his computations for the Rio Grande Arch in Costa Rica. They may be found in the *Engineering Record*, Vol. 16, page 434, for November 8, 1902, or in Skinner's "Details of Bridge Construction," Part I, page 78. The use of two symmetrical loads, as suggested in "Modern Framed Structures," serves a similar purpose.

The exact method of selecting an equivalent uniform live load for an arch span is explained on page 126 of the book last mentioned; but it will nearly always be satisfactory to take the equivalent uniform load for a simple span having a length of three-quarters of that of the arch span, and compute the stresses with the full panel loads derived therefrom.

The stresses in arches due to wind loads require careful analysis. "Modern Framed Structures" treats the subject quite well for ribs lying in vertical planes. The stresses in arch bridges with ribs lying in planes inclined to the vertical, so far as the author is aware, are not treated in any text-book; and the only discussion of the subject known to have appeared in print is one by J. Ensink, Esq., C. E., in *Engineering News*, Vol. 61, page 659 for May 21, 1910. That treatment handles the problem completely for the three-hinged, spandrel-braced arch. The general method there explained could be applied to the two-hinged and the hingeless arches, although the work of figuring the stresses in the arch members would be quite laborious. This inclining of the arches so as to bring their tops close together and to spread them below is an important feature of arch-bridge construction in that it adds greatly to the rigidity of the structure and reduces the total weight of metal. On the other hand, it adds a trifle to the pound cost of manufacture, but not enough to offset the saving in weight. In the author's 425-foot span of the Fraser River Bridge this expedient was adopted.

Any arch structure, especially when the ribs lie in vertical planes, shows a decided tendency to vibrate; hence the lateral bracing should be very rigid. In the spandrel-braced type there should be an upper lateral system connecting the top chords, a vertical bracing frame connecting each pair of opposite posts, and a particularly effective system between the bottom chords. In the three-hinged type the systems will have to be broken at the centre of the span. For the braced or the solid-rib deck arch-bridges there should be a top lateral system along the roadway, a vertical frame in the plane of each vertical, and effective bracing along both the upper and the lower chords of the arched ribs. Where the ribs rise above the floor there should be an effective system of bracing in the plane of the said floor, and also other systems along the top and the bottom members of the ribs, with efficient portals where the laterals are interrupted by the roadway clearance.

The combination of stresses in arch bridges is a little more complicated than it is in simple-truss spans, but less so than in trestles. The various stresses to be combined are as follows:

1. Dead Load.
2. Live Load.
3. Impact.
4. Wind.
5. Temperature.

For the summation of Nos. 1, 2, and 3, the usual intensities of working stresses are to apply. For that of either Nos. 1, 2, 3, and 4, or Nos. 1, 2, 3, and 5, the said intensities are to be increased thirty (30) per cent. Finally, for the summation of all five, the said intensities are to be increased forty (40) per cent.

As the question of the relative merits of the various kinds of arches is a much mooted one, and as but few bridge engineers seem to be posted thereon, it will be well to summarize concisely the preceding dissertation, even at the risk of wearying the reader by repetition.

1. *Hingeless Type.* Most rigid of all, but there is great ambiguity in the stresses, and the labor involved in making the computations is excessive. Should be adopted only when the abutments are very rigid, as any spreading of the foundations would be ruinous to the superstructure.

Claimed by some engineers to be the most economical of metal, but the correctness of the claim is not yet established.

2. *One-Hinged Type.* Very unusual, and possesses few advantages. Not quite as rigid as the hingeless type, and a little superior to it in respect to avoidance of ambiguity and to ease of computation.

3. *Two-Hinged Type.* Quite rigid, and involves considerably less stress-uncertainty and computation-labor than the two preceding types. Is much favored by most European and many American bridge specialists. Can advantageously be made three-hinged for dead-load stresses and two-hinged for live-load stresses.

4. *Three-hinged Type.* A little less rigid than the three preceding types, but avoids all ambiguity of stress distribution, and the method of stress calculation is simple. Undoubtedly preferred to all the other types by a majority of American bridge specialists.

5. *Cantilever Arch Type.* But little used up to the present. For certain localities might involve a small economy of metal. Probably not quite so rigid as an ordinary arch of like skeleton diagram. Worthy of more consideration by American engineers than it has yet received.

6. *Solid-Rib Type.* Can be used with any of the first four types. Suitable for all lengths of span, but becomes more uneconomical of metal as the length of opening increases. Has a fine appearance, hence is popular among designers.

7. *Braced-Rib Type.* Can be used with any of the first four types, and generally is more economic of metal than the solid-rib type, the longer the span the greater the economy. Can be made to produce an æsthetic appearance. Fairly popular among American engineers.

8. *Spandrel-Braced Type.* Can be used with Types 3, 4, and 5. Probably has some economic advantage over the solid-rib and the braced-rib types, especially for long spans, and is generally more rigid because of the greater depth of the arch girders or trusses. Has been used considerably in America.

Economy of Types. The number of arch bridges built up to the present time is comparatively so small, and the economic studies thus far made on arches have been of such an approximate character, that but little reliable information concerning the comparative economics of the various types is available. There is a general impression that the smaller the number of hinges the greater the economy, but there are conflicting opinions concerning that view. Again, it is probable, but not yet proved, that for exactly similar conditions of layout and loading a cantilever-arch bridge is a little less expensive than an ordinary arch with two flanking simple spans. Finally, the three standard kinds of ribs for fairly long spans in the order of their economy are generally supposed to be the spandrel-braced, the braced-rib, and the solid-rib types; but differences in the existing conditions at crossings may vary this order in certain cases.

For many years the author has been endeavoring to establish some approximate relation between the weights of metal per lineal foot for trusses and laterals of arch bridges and those for the corresponding simple truss bridges; but has met with very little success. He once submitted the question to his brother bridge specialists of America, but they were unable to throw any light upon the subject, because their opportunities to design and build arch bridges had been few and far between, and because the ratio of rise to span has a great effect upon the weight of metal in an arch. Of course, there is for any span length some economic value of that ratio; but it is not yet known, and it probably varies more or less not only with the span but also with the type of construction. The only practicable method of determining the original question would be to settle first that of the economic ratios of rise to span, design a few arch bridges with the said ratios, and make the comparison. This would have to be done for the solid-rib, the braced-rib, and the spandrel-braced types to make the job complete, adopting for the first set of curves the three-hinged type, and afterward modifying the results for the other three types of hinging. It is evident that the amount of work involved in such an investigation would be immense. It should be done by an experienced bridge designer, as the results would be worthless if obtained by any other investigator. The author suggests that one of his younger brother-specialists undertake the investigation. He certainly cannot spare the time to make it himself, the best that he can do being to offer the following records and the results of some special computations made for his 425-foot arch span over the Fraser River. Unfortunately that structure was designed according to the bridge specifications of the Dominion Government, which, as far as arches and other structures involving large reversing stresses are concerned, are wastefully extravagant of metal. As the author's office possesses no record of weight for the corresponding simple span designed according to those specifications, he has had to have the arch refigured according to his own so as to compare with his office diagrams of weights of simple-span, deck, railway bridges. The result of the special computation is that the weight of metal in the trusses, laterals, and floor system of the riveted arch span is to the weight of same in the corresponding simple-truss riveted span as seventy-five is to one hundred. It was found impracticable to make a fair comparison between the two cases without including the weights of metal in the floor systems. This showing is as favorable as possible to the arch; for the rise of the latter was about the economic amount, and the ribs were assumed to be battered, while in the deck simple span the trusses were in vertical planes. Had the trusses been battered, there would have been a small saving in the weight of the floor and possibly a little also in the lateral system. It is certainly a difficult matter so to adjust all the conditions as to give a fair comparison between the weights of metal for arches and truss spans in general; but, of course, in any particular case by taking due cognizance of all the factors, a correct comparison can be made. In the author's Hamilton, New Zealand, plate-girder arch of 340 feet span (See Fig. 26*i*), in which the ratio of rise to span is 0.124, the ratio of weights is 0.97; and in his Cambridge, New Zealand, spandrel-braced arch of 290 feet span (See Fig. 26*k*), in which the ratio of rise to span is 0.18, the weight ratio is 0.83. This great variation of ratios is due partially to the different types of construction and partially to the difference in the ratios of rise to span length.

In dealing with the comparative economics of arches and simple trusses, it must not be forgotten that there are other factors than mere weight of metal involved; for the pound price of the manufactured material is generally somewhat greater for the former, and sometimes the cost of erection also is larger. Again, the comparison of the costs of arch and truss superstructures alone is not of much importance, for an economic investigation to be of any value must include both substructure and superstructure; and the costs of the latter are likely to be very different in arch and simple-truss designs for any crossing.

For many years the author has been greatly desirous of establishing approximately correct formulæ for the weights of metal in arch ribs; and in 1907 he gave to one of his computers the task of preparing one for three-hinged, plate-girder arches with the special intention of having it appear for the first time in this book. The investigation was not fully completed at that time, and when the writing of this chapter was commenced the author gave to his assistant engineer, Mr. Hardesty, the task of preparing an entirely new study, which he did, finding quite satisfactory results. When he had finished the investigation for plate-girder arches, he was set the additional task of making similar calculations for open-webbed, riveted ones of the braced-rib type. The methods adopted in the computations are semi-rational and semi-empirical, approximate formulæ being derived for the length of arch, and percentages for weights of details being taken from the numerous records collected in the author's office during many years. On account of the unavoidable empiricism of the investigation, the formulæ are offered to the profession as mere approximations to exactness. They are fairly correct for average conditions of layout, but less so for unusual ones. The formulæ are derived directly for ribs of parabolic form, having the dead load uniform over the entire span; but they will serve equally well for ribs of other forms, provided that under dead load plus half of full live and impact loads over the entire span the moments in the rib near the quarter points are practically zero. Since this condition will hold for nearly every well-designed rib, the formulæ will apply in practically all cases. The weights given by them include those of the main sections, diaphragms, stiffeners, lacing, splices, pin-plates, pins, rivets, etc.; but do not include the weights of the skew-back pedestals.

In applying the formulæ, the moment and stress in the rib at the quarter point of the span are figured with full live plus impact loads on the far half of the span, and this is assumed to represent the average condition throughout the rib. In the open-webbed arch investigation it was assumed that the pins lie midway between the two chords, and that the web system is of the single-intersection Warren or Triangular type; but the formulæ can be applied to other forms with only slight error, certain of the coefficients being changed, if greater accuracy be desired, in order to suit the various special conditions.

The formulæ thus established are certainly sufficiently accurate for dead-load assumptions and even for preliminary cost estimates, but probably not for bidding by lump sum. While it is true that they were established for three-hinged arches, they can be used for arches with any condition of hinging, either by making the assumption that there is no material difference between the weights of arches of the various types of fixedness, or by multiplying the weights resulting from the formula by some assumed general ratio which the individual designer deems proper to employ in comparing the weights of three-hinged arches with those of the type of fixedness under consideration.

The formulæ for the three-hinged, plate-girder arch-ribs are as follows:

$$T = (2D + L + I)\frac{ln}{16}, \qquad \text{[Eq. 1]}$$

$$M = (L + I)\frac{l^2}{60} \text{ (foot-pounds)}, \qquad \text{[Eq. 2]}$$

$$A = \frac{T + \frac{2M}{d}}{p} + 9\,dt, \qquad \text{[Eq. 3]}$$

and

$$W = 4.8\,Ar; \qquad \text{[Eq. 4]}$$

in which l = length of span in feet,
h = rise of span in feet,
D = dead load per lineal foot of span per rib,
L = equivalent uniform live load per lineal foot of span per rib, taken for a loaded length of one-half of l,
I = impact load per lineal foot of span per rib, taken for the same loaded length,
T = total direct stress on section figured,
M = bending moment in foot-pounds at the same section,
d = assumed depth of arch rib in feet (depth of web or distance back to back of angles),
t = thickness of web or webs in inches,
A = average area of arch rib in square inches,
p = allowable unit stress from column formula,
$n = \sqrt{4 + \left(\frac{l}{h}\right)^2}$, to be taken from Fig. 26n,
r = ratio of length of rib to span length l, to be taken from Fig. 26n,
and W = weight of metal in rib in pounds per lineal foot of span.

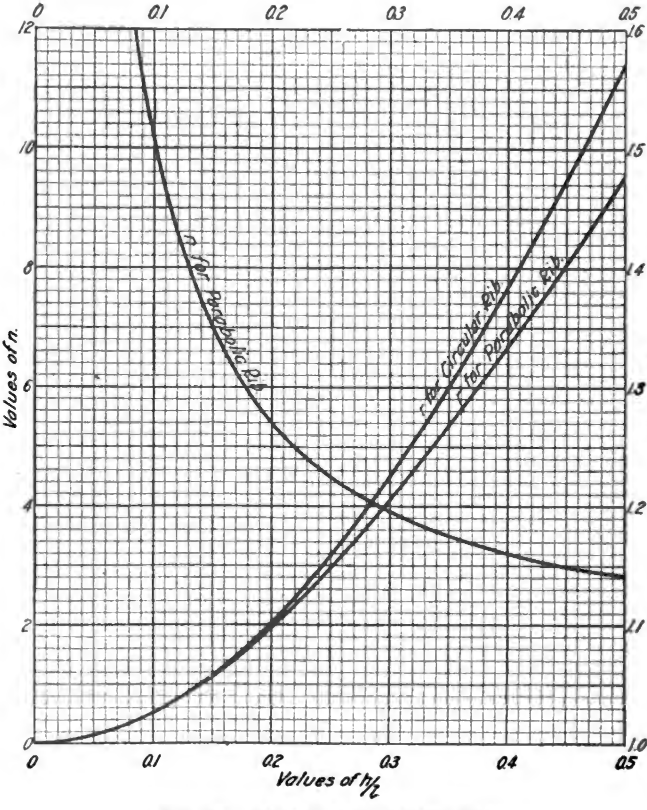

FIG. 26*n*. Values of *n* and *r* for Arch Ribs.

The formulæ for the three-hinged open-webbed, riveted arch-ribs are as follows:

$$T = (2D + L + I)\frac{ln}{16}, \qquad \text{[Eq. 5]}$$

$$M = (L + I)\frac{l^2}{60} \text{ (foot-pounds)}, \qquad \text{[Eq. 6]}$$

$$S = \frac{(L + I)\,l}{5}\left(1 - \frac{h^2}{l^2}\right), \qquad \text{[Eq. 7]}$$

$$A_c = \frac{T + \frac{2M}{d}}{p_c}, \qquad \text{[Eq. 8]}$$

$$A_w = \frac{2S}{p_w} \qquad \text{[Eq. 9]}$$

and

$$W = 4.6\,(A_c + A_w)\,r; \qquad \text{[Eq. 10]}$$

in which l, h, D, L, I, T, M, n, and r have the same values as in the case of the plate-girder arch, and

S = average maximum shear on all portions of the rib,
d = assumed distance in feet from centre to centre of chords,
A_c = average area of chords in square inches,
A_w = average area of web members in square inches,
p_c = average allowable unit stress in chords by column formula,
p_w = average allowable unit stress in web members by column formula,
and W = weight of metal in rib in pounds per lineal foot of span.

It was the author's intention to insert in either this chapter or an appendix the mathematical demonstrations of the manner of derivation of the various formulæ given above; but the computations involved are so lengthy that a consideration of space-economy forbids. Suffice it to say that the said mathematical work has been carefully checked by an independent computer and pronounced by him to be correct in principle and sufficiently close to exactness in all the assumed approximations. Moreover, the two formulæ for W have been tested by comparing their results when applied to the conditions of layout of the author's before-mentioned New Zealand plate-girder arch, Mr. Hodge's design for the proposed Harlem River arch bridge, and Mr. Schneider's design for a 720-foot arch span with the actually computed weights of those structures; and the agreement found was gratifyingly close. It was not practicable, however, to check against Mr. Buck's Niagara arch; because, unfortunately, the weights given therefor combine those for the ribs and those for the bracing.

Some seven years ago the author collected by correspondence some weight data on arch bridges from certain American bridge specialists. These are herewith reproduced in the hope that they may be of service to some reader in designing structures similar to those described.

F. C. Kunz, Esq., then Chief Engineer of the Pennsylvania Steel Company, sent the following information concerning the weights of metal in European highway and single-track railway bridges with arched ribs:

" HIGHWAY BRIDGES WITH ARCHED RIBS

"The weight g in pounds of the main trusses (including all wind bracing) for each foot of span (for two-hinged arches) is given by the formula,

$$g = Kb + 23.5z,$$

in which b designates the width of the bridge in feet,
z indicates the number of the main trusses,
and K is to be taken from the following table:

Span L, in feet............	33	66	100	130	165	195	230	260	295	330
Floor with ballast, K=...	6.5	12.7	19.2	26.3	34.3	42.6	52.0	61.2	71.4	83.6
Floor with double planking, K=.............	5.7	10.8	16.3	22.4	29.4	36.7	44.9	53.0	62.2	72.4

"For three-hinged arches the above values may be reduced fifteen per cent.

" RAILROAD BRIDGES WITH ARCHED RIBS

"The weight g of the main trusses and all bracings for single-track structures in pounds per lineal foot of span is given in the following table, the length of span L being expressed in feet:

For L=.........	33	66	100	130	165	195	230	260	295	330
g=.........	300	500	710	900	1,110	1,290	1,500	1,720	1,940	2,200

"For spans up to 230′ the above values correspond to the formula,

$$g = 100 + 6.1\ L.$$

These weights refer to two-hinged arches; for three-hinged arches a reduction of 15 per cent is admissible."

Attention is called to the fact that Mr. Kunz considers that three-hinged arches are lighter by fifteen per cent than two-hinged ones. This is contrary to the idea generally prevalent among American bridge engineers, and would indicate that European designers must provide liberally for ambiguity of stress.

C. C. Schneider, Esq., Past President of the American Society of Civil Engineers, and one of the most noted of American bridge specialists, wrote as follows:

> "I have data on hand of an arch bridge on which I figured recently, and I take pleasure in forwarding it to you herewith.
>
> "The span between centres of end-pins is 720 feet and rise 72 feet, or a ratio of rise to span of 1 in 10. The same consists of two arch ribs inclined. The distance between centres on top is 30 feet and on the bottom 52 feet, making a batter of 1 in 8.

"The bridge is to carry two trolley tracks, roadways, and sidewalks. The roadway with the trolley tracks is 36 feet wide, with a six-foot sidewalk on each side, outside of the roadway.

"The floor and its supports were designed in accordance with Cooper's Specifications of 1901, for Classes A1 and A2 Loading for City Bridges.

"The arch and its bracing were designed for a working load of 2,000 pounds per lin. foot, with the usual working stresses, in accordance with Cooper's Specifications; and also for a congested load of 4,000 pounds per lineal foot, allowing a maximum unit stress of 25,000 pounds per sq. in. for the combined dead and live loads and wind pressure.

"The weight of the steel superstructure is as follows:

Floor-beams and stringers	720,000 lbs.
Supporting columns with bracing	280,000 "
Lower chords of arch	1,200,000 "
Upper chords of arch	296,000 "
Web of arch	282,000 "
Lateral bracing in lower chords	260,000 "
Lateral bracing in upper chords	90,000 "
Sway bracing in arch	32,000 "
Total weight	3,160,000 lbs.

"The weight of the wooden deck is about 21,000 lbs. per panel."

Henry W. Hodge, Esq., the eminent bridge engineer, sent the following data for a design of his for an 825-foot arch bridge prepared for the City of New York.

PROPOSED ARCH OVER HARLEM RIVER, N. Y.

Span 825′ c. to c. of pins.
Rise 126′.
25 panels at 33′.
4 ribs, carrying equally, spaced 27′ c. to c. of chords.
Arch having pins at end only.
One 50′ roadway, two 15′ sidewalks; total width 80′.
Live load for floor and vertical posts 100 lbs. per sq. ft., or a road roller weighing 18 tons.

Live load for ribs 75 lbs. per sq. ft.	6,000 lbs. p. l. f. of bridge
Dead load,	
Roadway material, 100 lbs. per sq. ft. × 50′	5,000 lbs. p. l. f. "
Sidewalk material, 80 lbs. per sq. ft. × 30′	2,400 lbs. p. l. f. "
Metal in structure	13,200 lbs. p. l. f. "
Total dead load	20,600 lbs. p. l. f. of bridge

Weight of metal floor system	2,430,000 lbs.
Vertical posts	570,000 "
Bracing of vertical posts	644,000 "
Main ribs	5,828,000 "
Lateral and transverse bracing, main ribs	820,000 "
End shoes and pins	558,000 "
Total weight	10,850,000 lbs.

Highway unit-stresses are 20 per cent in excess of railway unit stresses.

Mr. Hodge at the same time sent the following data for the 840-foot-span arch bridge at Niagara:

NIAGARA ARCH

840′ c. to c. of pins.

20 panels at 42′ (panels in rib 21′ each, 2 to each vert. post).

150′ rise.

2 main ribs, each 26′ c. to c. of chords, 30′ c. to c. ribs at top and 69′ c. to c. at bottom.

One roadway 37½′, two sidewalks 4¼′ each = 46′ total width.

Live load 90 lbs. per sq. ft. for floor and vertical posts, 50 lbs. per sq. ft. or 2,300 lbs. p. l. f. for ribs.

Total dead load—

Timber flooring and railings	800 lbs.
Metal	4,500 "
Total	5,300 lbs. p. l. f.
Weight of metal floor system	766,000 "
Vertical posts and bracing	601,000 "
Ribs and bracing	2,057,000 "
Shoes and end pins	227,000 "
Total weight	3,651,000 lbs.

Frank C. Osborn, Esq., the well-known bridge engineer and steel inspector, sent the following data concerning some small highway arch-bridges designed and built by him.

"The Brooklyn Bridge is a braced spandrel arch and has a span of 168′ from centre to centre of end pins. The rise is 48′, and the width is 26′ from centre to centre of arch rings. Width of roadway 29′ 6″ with two sidewalks 6′ each. The loadings are as follows: Dead load 2,400 lbs. per lineal foot; live load 4,000 lbs. per lineal foot of bridge. The total weight of structural steel is 89 tons, which does not include the floor system.

"The Chagrin River Bridge is a plate-girder arch with a span of 168′ 9″ from centre to centre of ends. The rise is 27′ 6″, and the width 27′. The width of roadway is 32′ and no sidewalks. The loadings are as follows: Dead load, 2,600 lbs. per lineal foot, and live load, 3,000 lbs. per lineal foot of bridge. Total weight of structural steel is 87 tons. This does not include the floor system.

"The Riverside Cemetery Bridge is a combination of lattice and plate-girder work, the arch being of the crescent form, the middle half consisting of lattice work and the two end sections of plate-girder construction. The span is 142′ from centre to centre of end pins, and the rise is 24′ 6″ and the width 22′. The width of roadway is 17′ and the sidewalks 4′ each. The dead load is 1,400 lbs. per lineal foot, and the live load 1,500 lbs. per lineal foot of bridge. Total weight of steel is 30 tons."

In respect to the detailing of arch bridges there are only two points in which it differs essentially from that of simple-truss spans, viz., the crown and the ends. Where the latter are fixed, as in hingeless and one-hinged arches, much care will be required in designing the anchorage of the metal to the masonry. The anchor bolts should be figured for the greatest stress required to make the ends fixed under the most unfavorable conditions, and they should pass into the rock far enough and fasten thereto firmly enough to develop the full strength of their sectional area. The areas of the parts that bear upon the rock or upon the masonry of the abutments should be so large that the permissible pressures of the specifications given in Chapter LXXVIII will not be exceeded under any probable circumstances.

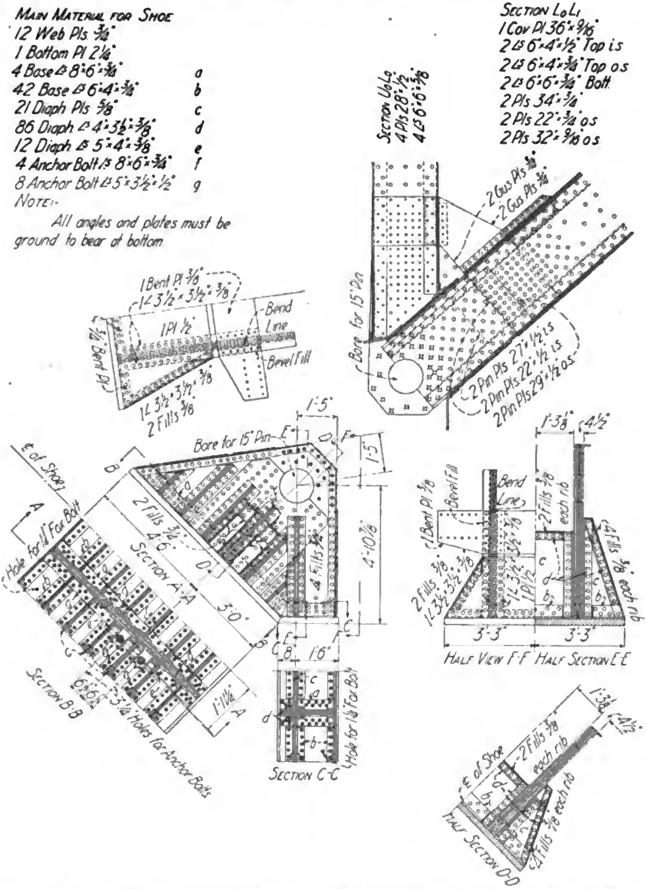

FIG. 26o. Pedestal Hinge for the Arch Span of the Canadian Northern Pacific Railway Bridge over the Fraser River.

The pedestals for hinged ends are generally not of complicated design. In Fig. 26o is shown one of those for the author's C. N. P. R., Fraser

River Bridge before mentioned. Neither are the crown hinges at all complex, as can be seen from Fig. 26*p*, which illustrates those for the same structure.

There is not much that is special to be said about the substructure of arches, for in many ways it does not differ essentially from that for simple spans. The abutments have no unusual features, except that the thrust upon them is inclined to the vertical and that care must be taken to design them so that they will distribute properly over sufficient area the concentrated loads at the skewbacks. Such abutments may be either single or divided into pedestals at the different points of bearing. They should be proportioned to carry not only the full load from the finished arch bridge, but also the various loads that may come upon them during construction, the latter for cantilever erection sometimes being vertical, and the former always inclined. As infinite pains must be taken to get all the pedestals located in exact position both horizontally and vertically, it behooves the resident engineer to build the substructure of any arch bridge with unusual care. In proportioning the pedestals or abutments the extreme variations of pressure thereon from various combinations of loading must be duly considered, taking special precaution to see that all effects of the wind are adequately provided for.

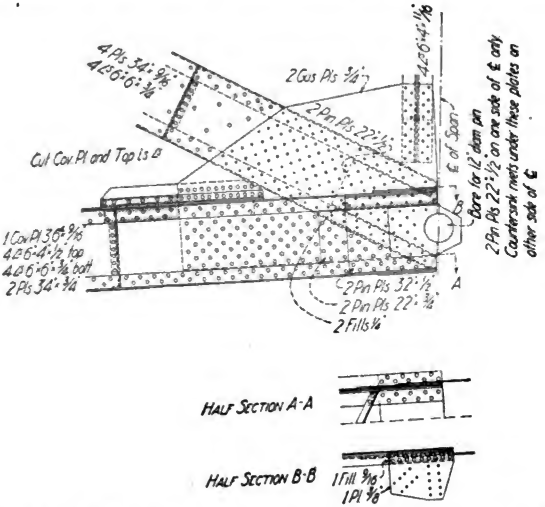

Fig. 26*p*. Crown Hinge for the Arch Span of the Canadian Northern Pacific Railway Bridge over the Fraser River.

CHAPTER XXVII

SUSPENSION BRIDGES

To very few American engineers does there ever come an opportunity to design a large or important suspension bridge; for that style of construction is but little used in this country, owing to the fact that, on account of its inherent lack of rigidity, it is not well adapted for carrying railroad trains, except in the case of very long spans, and because in highway bridge construction it cannot compete in cost with simple-truss, cantilever, or arch structures for ordinary crossings.

Although his first piece of bridge engineering, which came to him only a few months after graduation, was the mathematical investigation of compensating trusses for the avoidance of temperature stresses in suspension bridges by making the top chords of timber and the bottom chords of iron,* in his forty years of professional life the author has had occasion to build only half a dozen suspension bridges for light highway traffic and of comparatively short span, none of them exceeding four hundred feet, and to design only one long span railroad structure for a proposed crossing of the Hudson River, which structure, however, never materialized. On that account, much to his regret, he is unable to offer the reader a complete set of accurate diagrams or tables from which to compute quickly and easily the quantities of materials in any ordinary suspension bridge, as he has done in the case of simple truss spans, swing spans, and cantilevers, or even possibly to provide quite as accurate data as he has submitted for figuring the quantities of metal in steel arch structures. However, he will present a method of computing quickly the approximate quantities and costs for the superstructures of both railway and highway suspension bridges of ordinary types; and these will be sufficiently accurate for preliminary estimates of cost and for the determination of dead loads to be used in the more accurate computations which should be made in preparing actual designs. To give any figures of quantities or cost for the substructures of suspension bridges is absolutely out of the question, because they will depend entirely upon the local conditions at the proposed crossing. It is never a great task, though, for an experienced bridge engineer to compute the approximate cost of substructure for a bridge when all the governing conditions are known; and even the structural designer who is inexperienced in the practical features of bridge building, but who is well read generally on both theory and practice, can make a fair attempt at such computations.

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* This paper on "Compensating Trusses" was published in the 1876 *Proceedings* of the Pi Eta Scientific Society, now the Rensselaer Society of Engineers.

While the suspension bridge in some simple form has been in use for centuries, it has remained for the present generation of engineers to bring it to its highest development and to perfect, probably as far as is possible, the theory of its action. In the earlier types the floor was directly suspended by hangers to the main cables so that a load at any point would cause a further sagging or deflection in the cable at that point and a rising therein at other points. The effect of this was to produce a wave motion in the floor as the load proceeded across the span. This lack of rigidity prevented to a large extent the adoption of the suspension type where heavy live loads were to be carried. To overcome this defect the floor system was first modified by using long floor joists extending over several panels and adopting stiff and rigidly braced hand-rails, thereby distributing the load to a larger portion of the cable. This step was followed by the introduction of stiffening trusses running the full length of the span and attached at regular intervals (generally at each panel point) by hangers to the cables. The stiffening truss and its supporting cable form a redundant system; and in large structures it becomes an important matter to determine what portion of the load each carries. All the dead load is supported by the main cables; and the function of the stiffening trusses is to distribute a partial live load over the hangers so that each one, as nearly as may be, will transmit an equal portion thereof to the said cables.

Various factors affect this distribution of live load, such as the following:

1. Rigidity of the cable system. This is determined not by the elastic properties of the cable, but by the dead loads establishing a particular funicular polygon to which the cable conforms. A partial live loading tends to distort this polygon, and such distortion is resisted by the dead loads. The larger these dead loads the more rigid the cable system.
2. Rigidity of the stiffening trusses. This is determined by the elastic properties of the metal and the moment of inertia of the section of the trusses.
3. The position, length, and elastic properties of the hangers.
4. Changes in the length of the cables due to live loads or to variations of temperature.
5. The movement of the saddles or the deflection of the towers due to changes in length of the backstays from the same causes.
6. The variations in heights of towers due to changes in the tensions on the cables and backstays and also to differences in temperature.
7. The condition of the ends of the stiffening trusses, whether simply supported or anchored down.

8. The character of the stiffening trusses, whether continuous throughout or hinged at one or more intermediate points.

As the dead load is practically distributed uniformly over the span length, the funicular polygon assumed by the cable is a parabola defined by the equation,

$$y = \frac{4\,fx}{l^2}(l - x); \qquad \text{[Eq. 1]}$$

in which y = the ordinate measured downward from the x axis at a point distant x from the origin,
f = sag or versine of the cable,
and l = length of span.

The origin is taken at the saddle supporting the cable.

As the live load comes on the span, it produces a deflection in the truss which, in turn, through the hangers, tends to distort the funicular polygon previously established by the dead-load system, causing the immediate portion of the cable to drop slightly and the more remote portions to rise. The more flexible the truss the more pronounced this movement and the more variable the distribution of the load on the hangers. The accurate solution of this problem of distribution leads to long, involved, and difficult analysis, utilizing the physical properties of the materials, such as size, weight, areas, and elasticities. These are not all known at the outset, and some means of starting the analysis with fairly proximate assumptions or estimates is highly desirable. The method recommended by the author for making the first approximate determination of sizes and weights is based on the following assumptions, which have been the established practice for a long time.

1. That the curve assumed by the cable under all conditions of loading is sensibly a parabola.
2. That the stretch in the cable is relatively small and inappreciable.
3. That the ends of the stiffening trusses are free and that the trusses are rigid as compared to the cable system, no hinges therein being employed.
4. That the pull on the hangers is uniform, as the curve of the cable remains a parabola for any position of the live load.

Such assumptions lead to the following results:

If kl = portion of span covered by live load coming on at end,
n = number of hangers per truss,
w = live load per foot of truss,
W = live load on one hanger,
V = shear on truss,
and n_1 = number of hangers on the unloaded portion of truss,

then the load W on one hanger is,

$$W = \frac{klw}{n}. \qquad \text{[Eq. 2]}$$

The greatest shear for a particular loading will be at the head of that loading and will be equal to the load on one hanger multiplied by the number of hangers on the unloaded portion, or

$$V = \frac{klw}{n} n_1. \qquad \text{[Eq. 3]}$$

This becomes a maximum for the span when the live load occupies one-half thereof, for which position

$$V \text{ (max.)} = \tfrac{1}{4} wl \qquad \text{[Eq. 4]}$$

The maximum bending moment occurs when the live load covers 0.618 of the span, for which position it becomes

$$M = 0.0451 \, wl^2. \qquad \text{[Eq. 5]}$$

The section at which this maximum moment occurs is distant from the end of the span $0.382l$.

Let l_1 = the length of simple span which, for the same value of w, would give a moment at mid-span equal to the maximum moment in the stiffening truss, then

$$M = 0.125 \, wl_1^2 = 0.0451 \, wl^2, \qquad \text{[Eq. 6]}$$

$$\therefore l_1^2 = \frac{0.0451}{0.125} l^2, \qquad \text{[Eq. 7]}$$

and

$$l_1 = 0.6l. \qquad \text{[Eq. 8]}$$

It can be shown in a similar manner that a simple span six-tenths of the length of the stiffening truss will have an average shear approximately equal to that in the said stiffening truss.

If we assume, as Prof. Burr shows in his excellent book on "Suspension Bridges," page 39, that the maximum moment will extend over $0.236l$ at the centre and will diminish uniformly to the ends, the average bending moment for the entire span will be about sixty-two (62) per cent of the maximum, while in a simple span it is about sixty-seven (67) per cent.

The preceding relations afford a means of arriving at an approximate estimate of the weight of the stiffening truss. The impact to be provided for in the latter may logically be assumed as that given in either Fig. 7c, 7d, or 7e for a span of $0.6l$. If that impact is r, the total load for which the stiffening truss should be proportioned is $w(1 + r)$. In order to find the approximate weights of metal per lineal foot of span for the stiffening trusses, the author sought to utilize some of the diagrams of this treatise which give the corresponding weights for simple trusses; but he was unable to establish any satisfactory relation between the two, owing to the fact that in the simple span the top chords are polygonal and the truss depths are great, while in the stiffening trusses the chords are parallel and the truss depths are exceedingly small. On this account he has evolved the following method of determining the required truss weights:

Let us use the same nomenclature as before, and in addition thereto the succeeding:

s = intensity of stress in tension,
p = panel length,
d = truss depth,
θ = inclination of the diagonals to the vertical,
M_m = maximum bending moment on one truss due to live-plus-impact load, and
V_m = maximum shear due to same.

The following assumptions will be adopted:

First. The average moment for a truss with ends not anchored is 62 per cent of the maximum.

Second. The average shear for such a truss, when the effect of stress reversion is given due consideration, is 80 per cent of the maximum.

Third. The addition of 20 per cent for rivet holes to the area of the average tension member makes it strong enough to carry in compression, with due regard to the effect of l over r, a stress equal to that for which it was designed in tension.

Fourth. The proper percentage to add for details to the weight of any riveted stiffening truss is 40.

For the effects of vertical loads only we have the following:

The area of the average chord member will be

$$A_c = 1.2\left(\frac{0.62\,M_m}{ds}\right) \qquad \text{[Eq. 9]}$$

The area of the average vertical post will be

$$A_p = 1.2\left(\frac{0.8\,V_m}{s}\right) \qquad \text{[Eq. 10]}$$

The area of the average diagonal will be

$$A_d = 1.2\left(\frac{0.8\,V_m \sec\theta}{s}\right) \qquad \text{[Eq. 11]}$$

The corresponding weights of metal in one truss per lineal foot of span, exclusive of the details, will be, respectively:

$$Z_c = 2 \times 3.4 \times 1.2 \times 0.62 \times \frac{M_m}{ds} = 5.06\,\frac{M_m}{ds}. \qquad \text{[Eq. 12]}$$

$$Z_p = 1.2 \times 3.4 \times 0.8 \times \frac{V_m d}{sp} = 3.26\,\frac{V_m d}{sp}. \qquad \text{[Eq. 13]}$$

$$Z_d = 1.2 \times 3.4 \times 0.8 \times \frac{V_m \sec\theta \operatorname{cosec}\theta}{s} = 3.26\,\frac{V_m(p^2+d^2)}{sdp}. \qquad \text{[Eq. 14]}$$

The total weight of metal per truss will then be

$$\begin{aligned} Z_t &= 1.4\,(Z_c + Z_p + Z_d) \\ &= 1.4\left\{5.06\,\frac{M_m}{ds} + \frac{3.26\,V_m}{s}\left(\frac{p^2+2d^2}{dp}\right)\right\}. \end{aligned} \qquad \text{[Eq. 15]}$$

The next point to determine is the wind pressure per lineal foot of span on the structure. The intensity, or pressure in pounds per square foot of area opposed to the wind, is given by the formula

$$p' = 35 - \frac{l}{100}; \qquad \text{[Eq. 16]}$$

and the area per lineal foot of span receiving the wind pressure can be found with sufficient accuracy from Fig. 9c, entering it with a span length of $0.6l$. Calling the area thus found A makes the wind pressure per lineal foot of span to provide for

$$P = p'A. \qquad \text{[Eq. 17]}$$

The entire wind pressure will be assumed to be carried by the lateral system of the suspended span to the towers, thus ignoring any assistance that may be given by the cradling of the cables. In some cases this is correct, but in others it causes a small error on the side of safety.

Let b = the perpendicular distance between the central planes of trusses.

The moment from the wind pressure will be

$$M_w = \tfrac{1}{8} Pl^2; \qquad \text{[Eq. 18]}$$

and, assuming, for convenience in figuring, that the two lateral systems are combined into one, the maximum chord stress will be

$$S = \frac{Pl^2}{8b}; \qquad \text{[Eq. 19]}$$

and the average will be

$$S' = \tfrac{2}{3} S = \frac{Pl^2}{12b}. \qquad \text{[Eq. 20]}$$

For wind pressure the unit tensile working stress is thirty (30) per cent greater than that for live and dead loads; hence the average area of one combined chord will be

$$a = \frac{S'}{1.3\,s} = \frac{Pl^2}{15.6\,b\,s}. \qquad \text{[Eq. 21]}$$

To this must be added about twenty per cent to allow for rivet holes, making the gross area

$$a_1 = \frac{1.2\,Pl^2}{15.6\,b\,s} = \frac{Pl^2}{13\,b\,s}; \qquad \text{[Eq. 22]}$$

and the corresponding weight of steel per lineal foot is $\frac{3.4\,Pl^2}{13\,b\,s}$. Assuming the weight of the details to be 40 per cent of that of the main sections, the weight of one chord of the combined horizontal trusses will be

$$W_w = 1.4 \times \frac{3.4\,Pl^2}{13\,b\,s}. \qquad \text{[Eq. 23]}$$

As there are two chords, the total weight per lineal foot will be

$$2\,W_w = 2.8 \times \frac{3.4\,Pl^2}{13\,b\,s} = \frac{Pl^2}{1.37\,b\,s}$$

or, say for convenience, $= \dfrac{8\,Pl^2}{11\,b\,s}$. [Eq. 24]

Before combining Eq. 24 and Eq. 15, it will be necessary to substitute 1.3s for s *in the portion of the latter which pertains to the chords,* making

$$Z'_t = 1.4\left\{3.9\,\frac{M_m}{s} + \frac{3.26\,V_m}{s}\left(\frac{p^2 + 2d^2}{d\,p}\right)\right\} \qquad \text{[Eq. 25]}$$

The total weight of metal per lineal foot in the two trusses will be given by that one of the following two equations which indicates the larger value:

$$T = 2\,Z_t \qquad \text{[Eq. 26]}$$

or

$$T = 2\,Z'_t + 2\,W_w = 2\,Z'_t + \frac{8\,Pl^2}{11\,b\,s} \qquad \text{[Eq. 27]}$$

For railway bridges, the weight F per lineal foot of span for the floor system is easily ascertained from the proper diagram of Chapter LV, and the weight L per lineal foot of span for the lateral system can be found therefrom either directly or by extension; but for highway bridges it will be necessary to figure these values directly.

The total weight per lineal foot of span for both the metal and the flooring of track (F_t) will be

$$W_1 = T + F + L + F_t. \qquad \text{[Eq. 28]}$$

To this will have to be added W_s, the approximate weight per lineal foot of span for the suspenders, and an assumed weight W_c per lineal foot of span for the cables.

The total load per lineal foot for computing the stresses in the latter will, therefore, be

$$W_c' = W_1 + W_s + W_c + 2w\,(1 + r'), \qquad \text{[Eq. 29]}$$

r' being the coefficient of impact for a span l as given in one of Figs. 7*c*, 7*d*, and 7*e*. This is for all of the supporting cables and must be divided by n_2, the number of cables, in order to obtain the total load per foot W_2 per cable for computing the size. The values of W_s and W_c will have to be first assumed and then computed, but the work involved in so doing is quite small, especially as it is likely that one or two trials will suffice to determine the sizes and weights with sufficient accuracy.

It must not be forgotten that the greatest load on the hangers will occur when the span is fully loaded and not when the greatest concentration comes at the panel point in question, as is the case for ordinary truss spans. Again, the impact to use in figuring the hanger stress is that for the whole span, l, and not that for two panel lengths of the truss, as is customary in ordinary bridge designing.

If l' is the panel length, or distance between consecutive hangers, the load per hanger for each cable will be $W_2\,l'$. This may be used in laying off a force diagram and an equilibrium polygon to coincide with the cable, from which the tension at any point in the latter is readily scaled. Or the total load per cable may be expressed in pounds per lineal foot of span and the following well known formula used for finding the stress in the said cable:

$$T_c = \frac{W_2 l^2}{8f} \sec \alpha, \qquad \text{[Eq. 30]}$$

where α = angle that the tangent to the curve makes with the horizontal axis at the point considered, and f = deflection of cable.

At the point where the cable is supported the tension becomes

$$T_o = \frac{W_2 l^2}{8f} \sqrt{1 + \frac{16f^2}{l^2}}. \qquad \text{[Eq. 31]}$$

Dividing this by the allowable unit stress gives the required section of cable, from which an estimate of weight may readily be made.

Up to this point the assumption has been that the stiffening trusses are free to rise at their ends; but if they are anchored down, the figures for bending moments and shears will be somewhat different; and, consequently, the weight of metal in the trusses will be changed.

The maximum positive and negative bending moment for ends anchored, as shown by Prof. Burr and other writers, is given by the equation,

$$M = \frac{wl^2}{54}. \qquad \text{[Eq. 32]}$$

The moment to be provided for, allowing for the effect of the reversal of stresses, will then be

$$M_m = \frac{1.75\, wl^2}{54} = \frac{wl^2}{31} = .0323\, wl^2 \qquad \text{[Eq. 33]}$$

Let l_2 be the length of simple span which, for the same load per lineal foot, will give the above moments; then

$$M = \frac{wl^2}{31} = \frac{wl_2^2}{8}, \qquad \text{[Eq. 34]}$$

and

$$l_2 = 0.5\, l. \qquad \text{[Eq. 35]}$$

The equivalent span length for shear, however, will be about six-tenths of that of the stiffening truss, the same as in the case of the span with free ends.

The maximum moment occurs at the third points, hence the chords of the middle third of the span will have to be of uniform section. If, as in the previous case, the moments be assumed to vary uniformly from the points of maximum to zero at the ends, the average moment will be 0.67 of the said maximum, or exactly the same as in the simple span.

The methods of finding the weights of metal for stiffening trusses with anchored ends are the same as those in the case of trusses with free ends, except that in this case the average moment is 67 per cent of the maximum, as was just stated, and that the average shear for which the web members are to be designed, after allowing for the effect of stress reversal, is 1.6 times the maximum shear. We then get, instead of Equation 15, the formula,

$$Z_t = 1.4 \left\{ 5.47 \frac{M_m}{d\,s} + \frac{6.52\ V_m}{s} \left(\frac{p^2 + 2d^2}{d\,p} \right) \right\} \qquad \text{[Eq. 36]}$$

Although the numerical coefficients within the brackets of Eq. 36 are larger than the corresponding ones of Eq. 15, the resulting value of Z_t is smaller, because in Eq. 36 the value of M_m is only .0323 wl^2 instead of .0451 wl^2, and V_m is exactly one-half of what it was formerly, viz., $\frac{1}{8}\,wl$ instead of $\frac{1}{4}\,wl$. From this it will be seen that the anchoring of the ends of the span saves between 20 and 25 per cent of the weight of the chords of the trusses, but has no effect at all on that of the webs.

The corresponding change in Eq. 25 will give

$$Z'_t = 1.4 \left\{ 4.21 \frac{M_m}{d\,s} + \frac{6.52\ V_m}{s} \left(\frac{p^2 + 2\,d^2}{d\,p} \right) \right\} \qquad \text{[Eq. 37]}$$

Substituting the values of Z_t and Z'_t given respectively in Eq. 36 and Eq. 37 will indicate the proper value of T to use when finding the total weight of metal per lineal foot of trusses.

It will be noted that the method outlined applies to trusses of nickel steel or other alloy steel, as well as to those of ordinary carbon steel; for, the value of s can be chosen to suit the material used. It thus makes it possible to determine in a short time what kind of steel is best suited for any structure. In this comparison there must, of course, be taken into account the changes in the sections of the hangers and cables due to the changes in the weights of the stiffening trusses.

No attention has been paid to the condition of intermediate hinges in stiffening trusses, because the author does not see any real advantage in using them; for while they may effect a certain saving of metal in the trusses, they certainly cause a loss of rigidity in the structure. It would be bad policy on the part of the designer to sacrifice even a small portion of the none too adequate stiffness of a suspension bridge in order to make a small reduction in its cost.

The economic depth for stiffening trusses has been determined theoretically by Dr. D. B. Steinman in his excellent little book on "Suspension Bridges and Cantilevers." He finds that it is about one-fortieth ($\frac{1}{40}$) of the span, and states that this is somewhat higher than the average of past practice, probably because most designs have been a compromise between the demands of economy and those of æsthetics. In his opinion, the Williamsburg Bridge, which is the only long-span structure conforming to this economic ratio, is marred in appearance by the excessive depth of the stiffening trusses. This limit does not strike one as being very high, though, considering the fact that the trusses are generally through ones and that they must provide a clear headway ranging between twenty and twenty-five feet. If twelve hundred feet be taken as the minimum span length for which it would be legitimate to consider the adoption of a suspension bridge for railway traffic, the economic truss depth would be thirty feet, which is a little shallower than it is practicable to adopt for a double-track bridge with floor-beams and portal bracing that have ample depths for rigidity. A serious objection to employing shallow deck stiffening trusses is their unsightly appearance. All things considered, it is generally advisable to make the truss depth as shallow as the governing conditions will allow, provided that the economic depth be not varied from too radically.

The inferior limiting ratio of distance between central planes of stiffening trusses to span length has not yet received due attention by engineering writers. The author is of the opinion that it ought to be about as one is to thirty, that for simple spans being as one is to twenty. There are two good reasons for placing a minimum limit to this ratio, viz., to avoid vibration and to make the various compression chords, which, in a way, form one long strut of the same length as the span, have a reasonable ratio of length to radius of gyration. For the thirty-to-one limit the ratio of length to radius of gyration would be about sixty, which is well within the bounds of good practice in strut proportioning.

The economic cable rise for many years has been recognized as varying from one-tenth to one-eighth of the span length. For bridges in which the side spans are without suspenders, Dr. Steinman finds that it is about one-ninth of the main span, and for those in which the side spans are suspended from the backstays it is about one-eighth thereof.

The greatest practicable span length for suspension bridges is a subject to which much attention has been devoted of late years. In 1894 a special board of officers of the United States Engineer Corps was appointed to investigate the question; and they found a limit of 4,335 feet, but the assumptions made were rather arbitrary. Dr. Steinman finds that the limit varies from 3,500 to 4,900 feet, depending upon the assumed live load, which he fixed within the limits of 10,000 and 20,000 pounds per lineal foot of span. Defining the limiting economic span as that at which the revenue from traffic just balances the annual cost of the structure, interest and everything else being included, Dr. Steinman finds that the limit is 3,170 feet. But with the advent of a high alloy steel for the stiffening trusses this limit, as well as the extreme practicable constructive limit, will be increased—possibly about in the proportions found by the author for cantilevers in his paper on "The Possibilities in Bridge Construction by the Use of High Alloy Steels."

In order to make clear to the reader the method herein described for finding the approximate weights of metal per lineal foot of span in riveted stiffening trusses, etc., by using the various diagrams of this treatise and the directions given in this chapter, the following example for a double track railway bridge is offered:

Span length.................................... 1,200 feet

Live load	Class 50
Distance c. to c. of trusses	40 feet

Ends of trusses free, *i.e.*, not anchored to masonry.
Metal all carbon steel.

Assume that there are thirty-six panels of thirty-three and a third feet each, and that the minimum truss depth practicable is thirty-three and a third feet, giving the diagonals an inclination of forty-five degrees.

Referring to Fig. 55*x* and taking a span length of 800 feet, which involves a width of 40 feet between central planes of trusses, the weight of metal in the floor system is found to be 1,500 pounds per lineal foot; and an extension of the line for weight of lateral system to 1,200 foot spans makes the weight 1,440 pounds per lineal foot, but this must be reduced for the smaller width to about 1,200 pounds, as per the directions given in Chapter LV.

From Fig. 6*e* we find that the live load per truss for a span of 720 feet (0.6*l*) is 5,400 pounds per lineal foot, and from Fig. 7*c* that the impact for a double-track railroad bridge of that span is ten (10) per cent, making the total live load per truss = 1.10 × 5,400 = 5,940, say 6,000 pounds; and, of course, there is no dead load to add.

The weight of metal per lineal foot per truss is found by substituting the preceding values in Eq. 15, giving

$$Z_t = 1.4 \left\{ 5.06 \times \frac{.0451 \times 6{,}000 \times (1{,}200)^2}{33.3 \times 16{,}000} + \frac{3.26 \times 6{,}000 \times 1{,}200}{4 \times 16{,}000} \times (1 + 2). \right\}.$$

$$= 1.4(3{,}697 + 1{,}110) = 6{,}700 \text{ lbs. nearly.}$$

The intensity for the wind pressure on a 1,200-foot span is $35 - \frac{1{,}200}{100} = 23$; and from Fig. 9*c* we find by extrapolation that the area of trusses per lineal foot on a double-track railroad bridge of 720 feet span is 37.6 square feet, and that on the floor is 7.6 square feet, making a total of 45.2 square feet, which multiplied by 23 makes 1,040 pounds per lineal foot.

The average extra weight of chord metal per lineal foot for resisting wind pressure, according to Eq. 24, will be

$$2\,W_w = \frac{8 \times 1{,}040 \times (1{,}200)^2}{11 \times 40 \times 16{,}000} = 1{,}700 \text{ lbs.}$$

Substituting in Eq. 25 gives

$$Z'_t = 1.4 \left\{ 3.9 \times \frac{.0451 \times 6{,}000 \times (1{,}200)^2}{33.3 \times 16{,}000} + \frac{3.26 \times 6{,}000 \times 1{,}200}{4 \times 16{,}000} \times (1 + 2) \right\}$$

$$= 1.4\,(2{,}844 + 1{,}110) = 5{,}536 \text{ lbs.}$$

$$T = 2\,Z_t = 2 \times 6{,}700 = 13{,}400$$

and $T = 2\,Z'_t + 2\,W_w = 2\,(5{,}536 + 1{,}700) = 14{,}472$ lbs.

Adding to this the weight of metal in the floor and lateral systems gives 17,172 lbs. as the total weight of metal per lineal foot in the span. To this should be added about 800 lbs. for the two tracks, making about 18,000 lbs. as the dead load, or 9,000 lbs. per lineal foot per truss.

The live load per lineal foot per truss for a 1,200-foot span, as per Fig. 6*e*, is about 5,300 pounds, and the impact for that span length, as per Fig. 7*c*, is eight (8) per cent., making the total live load about 5,700 pounds. Adding to this the dead load just found makes 14,700 pounds as the total load per lineal foot per side. The load on one hanger will, therefore, be $14{,}700 \times 33.3 = 485{,}000$ pounds.

Combining the live load over the full span with the weight of the stiffening trusses, the floor and lateral systems, the track, the hangers, and the cables, the total load carried to the towers is ascertained. The vertical component of the tension in the backstays is to be added to that from the cables. The towers are to be designed as columns subject to bending, if movement of saddles is not provided for, or if the lower ends of their steel columns be not hinged. These towers may be built as braced steel bents resting on masonry piers, or as masonry columns or shafts. The question of adopting fixed or hinged ends for the feet of steel tower-posts is still an unsettled point. The author favors the hinged ends in order to avoid large bending moments on the columns from eccentricity of loading. He once designed a wooden tower for a highway suspension bridge and figured on hinged ends; but during his absence from the office, while the detail plans were being made, the chief draftsman upon his own responsibility took the liberty of improving (?) the design by changing to fixed ends. A few years after the erection was completed the columns got a little off the vertical, due, probably, to some slight stretch in the backstays, and some bending resulted. Most of the trouble was corrected by an adjustment that had been provided in the anchorage; but the author was glad that, when in due time it became necessary to renew the timber, he was able to substitute metal therefor and to put in hinged ends at the feet of the four columns.

The backstays must be attached to anchorages which have unyielding foundations, such as rock. With any less stable foundation material, a certain amount of movement or displacement is apt to occur when the anchorage is subjected to the pull of the cable. Any displacement of one or more supports causes disturbances in the distribution of stresses in the span.

The preceding approximate method of figuring suspension bridges will enable the designer to arrive at a provisional estimate of cost and will also furnish data, such as weights, areas, and moments of inertia, needed for the more accurate methods of analysis. For such methods, the reader is referred to "Modern Framed Structures," Part II, by Johnson, Bryan, and Turneaure; "Suspension Bridges," by Prof. Wm. H. Burr; and to

"Melan's Theory of Arches and Suspension Bridges," translated by Prof. D. B. Steinman.

That even with the best possible system of analysis there is considerable ambiguity of stresses in the stiffening trusses of suspension bridges goes without saying; but it is unavoidable. If it be properly anticipated and provided for according to the latest and most approved theory, it will do no great harm. It has been rumored that the crippling of one of the trusses of the Brooklyn Bridge a few years ago was due to this cause; but, of course, when it was designed, over forty years ago, there was not as much known about the computation of stresses and the science of proportioning as there is at the present time.

There are only two conditions in this country that call for the adoption of a suspension bridge. The first is at a wide, high crossing of a river or gorge where a cheap highway bridge is needed, and even in such a case careful figuring should be done so as to see whether some more simple style of structure cannot be built for less money. While it is true that the suspension bridge can be erected almost entirely without falsework, it must be remembered that the cantilever and the arch do not require very much, and that simple spans can be constructed by semi-cantilevering, thus avoiding excessive expense for temporary supports. If any other type than the suspension bridge can be built for such a highway crossing at about the same cost as the latter, the more rigid structure should be adopted, because light suspension bridges are very vibratory; and, moreover, they do not resist wind pressure as well as the other bridges, because of the fact that there is often nothing but the weight of the floor alone to prevent an uplifting wind from destroying the structure, or, at least from injuring it so materially as to put it out of commission for some time. The second condition referred to is that of a span so long as to make the suspension type so much cheaper than the cantilever that it has to be adopted. The question of the comparative economy of cantilevers and suspension bridges is treated at length in Chapter LIII, to which the reader is referred for further information.

The longest suspension bridges in the world are the three over the East River uniting New York City proper and Brooklyn. The longest of them is the Williamsburg Bridge with a span of 1,600 feet, and next comes the old Brooklyn Bridge, which is only four and a half feet shorter, and then follows the Manhattan Bridge with a span of 1,470 feet. There are two other suspension bridges in the United States and one in Mexico having spans exceeding one thousand feet in length, but, as far as the author can ascertain, there are no others in the world that come up to the latter limit, the next longest being the one over the Danube at Buda-Pesth with a span of 951.5 feet. In Merriman and Jacoby's "Roofs and Bridges" and in Merriman's "American Civil Engineers' Pocket Book" will be found a list of the twenty-one longest spans in America.

Some longer suspension-bridge spans than these have been contemplated and even computed with more or less detail—mainly several projected structures to cross the North River at New York City. Messrs. Geo. S. Morison, Gustav Lindenthal, and Henry W. Hodge have made designs for that crossing; and it is not at all unlikely that the last-mentioned engineer and his financial associates in the not very distant future will succeed in consummating the enterprise. For the sake of the engineering profession as well as for other good reasons, it is to be hoped that they will be successful. The building of such a structure as the one they contemplate would be a fitting climax to an already brilliant professional career.

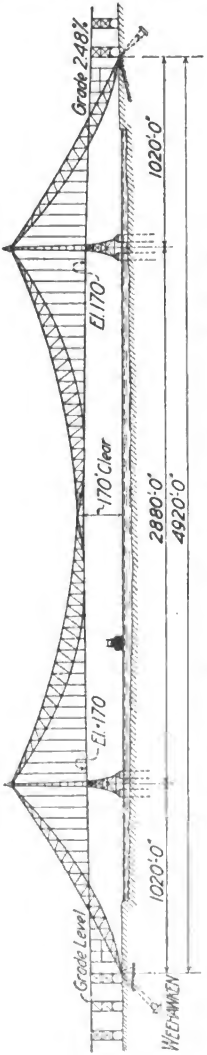


FIG. 27a. Proposed Suspension Bridge over the North River, New York City.

In Fig. 27*a* is shown a layout of the proposed structure. From this it will be seen that the main span is to have a length of 2,880 feet, and each flanking span one of 1,020 feet, making the total length of the bridge proper (*i.e.*, exclusive of the approaches) 4,920 feet. Including the approaches, it is 8,330 feet long. The maximum grade is 2.48 per cent. The vertical clearance above the water is 170 feet, and the total height of the towers above the same is about 630 feet. Each half of the main span and each anchor-arm are composed of four crescent-shaped trusses, the chords of which are the suspension cables, consisting entirely of eye-bars, and the webs of which are rigid members. The layout is so made that the tension in the cables will always exceed the compression on the chords of the trusses, rendering any stiffening of these members unnecessary. It is intended to make these eye-bars of some alloy steel that will give a minimum elastic limit of 80,000 pounds per square inch in the finished bars. Mr. Hodge counts upon being able to obtain such metal by paying, if necessary, an exceedingly high price for it; but if some experiments on vanadium steel that the author has in contemplation prove successful (as he thinks they will, if they can be made), the desired eye-bars will be procurable at a very reasonable cost.

The following are extracts from Mr. Hodge's report of March 18, 1913, to the New York State Bridge and Tunnel Commission concerning the proposed structure.

"We have assumed that the bridge should have a capacity for eight lines of rapid-transit trains, there being two tracks for subway trains, two for elevated-railway trains, two for New York (slot) trolley cars, and two for New Jersey (overhead) trolley cars; and in addition thereto we have provided two driveways, each 36 feet wide and having a capacity for four vehicles abreast, and there will be two sidewalks, each 8 feet wide, giving a splendid point of view for pedestrians during naval parades and the river pageants. The total moving load of all lines of travel, when simultaneously loaded, is assumed to be twenty thousand pounds per lineal foot of bridge.

"We have assumed the clear height over the river to be 170 feet, which is 35 feet higher than any of the bridges over the East River, and this will require a grade of only 2.48 per cent from Ninth Avenue to the New York pier, and thence level to the Hudson County Boulevard Loop in Weehawken. The clear distance between the pier-head lines of 1897 at this site is 2,730 feet, making a span of 2,880 feet from centre to centre of towers, which will make the longest span in the world, but thoroughly practical from the points of construction, erection, and expense. The total length of the bridge, including approaches, is 8,330 feet from Ninth Avenue, New York, to the Boulevard in Weehawken. . . . . . .

"A suspension type has been adopted as being the most practical and economical for this type of span.

"The towers will be of steel, reaching a height of about six hundred feet above the water. The main cables will be stiffened with secondary cables and web members, and the anchorages will be carried into the natural ledge rock on each shore.

"All lines of travel will be on the same level, thus allowing passengers from any train or trolley car readily to transfer to any other line in case of a block. The total width of the floor will be 204 feet, and it will have a greater capacity than any bridge yet constructed.

"The real estate required for the approaches will not be very expensive, as on the New York side the land in this vicinity is largely unoccupied, and in New Jersey there will only have to be secured an easement to pass high over the West Shore Railroad freight-storage tracks.

"In New York it is proposed to buy the land between two cross streets from the river to Ninth Avenue, thus avoiding all abutting damages, and after the structure is completed the greater portion of this land can be used for playgrounds, public buildings, warehouses, or any other class of buildings. The value of the necessary real estate has been arrived at by increasing the assessed valuations by the usual allowances. The total estimated cost of the entire undertaking is as follows:

"River span and anchor spans	$27,100,000
Approaches	1,900,000
Total structures	$29,000,000
Engineering and contingencies	2,900,000
Real estate	4,800,000
Interest during construction	5,300,000
Total cost	$42,000,000

"Construction on this type of structure can be carried on at many points simultaneously, and we estimate that the entire work can be completed in six years from the time work is commenced."

In October, 1896, the late Geo. S. Morison, Past President of the

American Society of Civil Engineers, presented to that society a paper entitled "Suspension Bridges—A Study," and it was discussed by Messrs. T. C. Clarke, Joseph Mayer, Francis Collingwood, Theodore Cooper, E. Gybbon Spilsbury, and W. H. Breithaupt. The paper, like all of Mr. Morison's writings, is both able and valuable to the engineering profession, as are likewise the discussions thereon. It will pay well any one interested in the subject of suspension bridges to read thoroughly both the memoir and the discussions. The basis of the paper is a study for a bridge to cross the North River, the main opening thereof being 3,080 feet, and the backstays being unloaded. The live load was 11,000 pounds per lineal foot over the entire structure, or but little more than half of that for Mr. Hodge's later design. The essential feature of the Morison design was that the cables were not to be built of straight wires but of wire ropes, socketed at the tops of the towers and in the anchorages. The ends of the stiffening trusses were to be anchored down, and there were to be four cables. Mr. Morison estimated that it would require five years to complete the structure.

Mr. Lindenthal's design for this crossing was made in the late eighties, and was described by him in a paper, entitled "The Economic Conditions of Long-Span Bridges with Special Reference to the Proposed North River Bridge at New York City," and read before the American Association for the Advancement of Science. It was published in *Engineering News* for November and December, 1889.

CHAPTER XXVIII

MOVABLE BRIDGES IN GENERAL

MOVABLE spans are required in bridges crossing navigable streams when they are not high enough to provide proper clearance for passing vessels. Before taking up the subject of movable structures, it will be well to consider the relative advantages and disadvantages of high and low bridges for the crossing of great rivers. As a rule, there is very little difference in the first cost of a high and of a low bridge for any such crossing, what little there is generally being in favor of the latter and seldom amounting to more than ten per cent. Each pier of a low bridge is cheaper than the corresponding pier of a high bridge; but this saving is offset by the cost of the pivot pier, which is extra. The superstructure of a low bridge may be a trifle lighter than that of the corresponding high bridge, but the more expensive metalwork of the draw span generally overbalances this. It is in the low, short trestle-approaches that the low bridge costs less than the high one. As these approaches are generally built of untreated timber, they have to be renewed about once in every eight years, and the cost of their renewal is a regular fixed charge, which lessens the annual net income from the bridge. Herein lies the superiority of the low bridge for such crossings. Nor is this its only advantage; for, by its adoption, there is avoided considerable climb at each end of the structure. On the other hand, the low bridge involves some expense for operation in excess of that for the high bridge, which is quite an important matter when there is much river traffic, but which is of slight importance when the draw has to be opened only a few times per season, as is the case with bridges over most western navigable streams. Everything considered, whenever there is any choice between a high and a low bridge, especially when the stream does not carry much traffic, the author favors the low bridge, not so much because of its smaller first cost, but mainly on account of the less expense required for maintenance.

The history of movable bridges goes back into the dim and distant past, for bascules were used over the moats that surrounded castles during the Dark Ages, and the pontoon bridges of the Romans undoubtedly had portions that could be removed in order to permit the passage of vessels.. It was not until the advent of timber trusses that it became possible to build structures across navigable streams of some size, and then arose the problem of providing a passageway for both vessels and bridge traffic. Bascules operated by hand power were first employed for this purpose, but as they were necessarily limited to very small openings, the next step was the evolution of the swing with either a pivot or a turntable; and when iron and steel took the place of timber, it was natural that the wooden rotating draw should be copied in metal. For many years the swing bridge served its purpose excellently, and even to-day it is still the most common kind of movable span; but with the advent of great business on the waterways its defects became apparent. In narrow channels the obstruction of the stream by the pivot-pier and the draw protection is a serious matter as far as navigation is concerned, and in many cases it affects materially the hydraulic regimen. Again, the time required for opening and closing a swing in a crowded city is far greater than the populace is willing to submit to without protest. Besides, the dock front adjacent to a rotating draw is not available for business. On these accounts the various kinds of lift bridges were evolved.

In this and the three following chapters, all of which deal with movable bridges, the treatment has been made general and descriptive; for, as explained in the Preface, it has been arranged that the author's former partner, Mr. Harrington, is soon to write in detail a complete and exhaustive work on the subject of "Movable Bridges." It is mainly for this reason that these four chapters on the subject do not illustrate any details.

Movable bridges may be divided into the following classes:

1. Ordinary rotating draws.
2. Bob-tailed swing spans.
3. Horizontal-folding draws.
4. Shear-pole draws.
5. Double, rotating, cantilever draws.
6. Pull-back draws.
7. Trunnion bascule-bridges.
8. Rolling bascule-bridges.
9. Jack-knife or folding bridges.
10. Vertical lift-bridges.
11. Gyratory lift-bridges.
12. Aerial ferries, transporter bridges, or *transbordeurs*.
13. Floating or pontoon bridges.

The ordinary rotating draws will be discussed at length in the next chapter.

The bob-tailed swing span is a variation of the ordinary rotating draw formed by shortening one of the arms and counterweighting it so as to balance the structure about the two principal vertical planes containing the axis of rotation. It is not a common type of construction because of the objectionable feature of unbalanced wind loads, to which it is generally subjected. It is needed in those localities where the pivot pier is at or near one bank and where the shore arm, if of the usual length, would interfere with buildings or prevent the use of valuable property. As far as the cost of construction is concerned, there can be but little, if any, economy in its employment; for the extra cost of the machinery necessitated by the unbalanced wind load added to the cost of the counterweights must offset the net saving in cost of superstructure due to the shortening of a moving arm and the corresponding lengthening of the adjacent approach.

The horizontal-folding draw is such an objectionable style of railway bridge construction as hardly to merit even a passing notice. It is, of necessity, applicable to only very short spans. It consists of a pair of girders spaced about five feet centres, with the rails attached directly to the top flanges, stayed at intervals by hinged struts like a parallel ruler, each girder being hinged at one end to the abutment upon which it rests, with the other end tied back to a short tower. Such an arrangement permits the girders to revolve laterally nearly ninety degrees, one bearing being located in advance of the other so as to make such a large rotation possible. Blocks are used under the outer ends of the girders to receive the live load reaction and thus prevent any moving load effect upon the tower. It would be difficult to design a structure more crude or unsatisfactory than this; and yet it is said that there are still many such bridges scattered throughout the New England States. In addition to its general loose-jointedness this type of movable span has the exceedingly dangerous feature of being wholly without track ties or real lateral bracing of any kind. What would happen to both it and the train in case of derailment of passing wheels would not be at all difficult to prognosticate! The unstiffened condition of the top flanges of the girders is a violation of an important requirement in scientific bridge designing. In case of a wreck involving either the loss of human life or personal injury, caused by a structure of this type, the jury should certainly find the railroad officials guilty of criminal carelessness for permitting such a glaring breach of safe construction to remain on their road.

The shear-pole draw is somewhat similar to the horizontal-folding draw, but is not quite so objectionable, as it permits of the use of a floor. It has a single leaf turning around a pivot at one end, the other end, while swinging, being suspended from the top of a two-legged shear-pole by rods which are connected to a pivot that lies directly over the pivot below. The shear-pole is stayed by guy rods. When the bridge is closed it forms a simple span supported at both ends. The employment of this type of opening span is not to be recommended.

Very few double, rotating, cantilever draws have yet been constructed. There is one, built many years ago, across the canal at Cleveland, Ohio; and a number of them at various times have been suggested and figured upon, including once a large one by the author. The advantages claimed for this type of structure are a wide waterway and the retreating of either span without serious injury when struck by a vessel before it is fully opened; while its disadvantages are excessive first cost, ambiguity of live load stress distribution, and the double cost of operating two independent spans. It is recognized, of course, that when electricity is used as the motive power, both spans can be operated by one man by means of a submerged cable; but in no case is it advisable, on account of prudential reasons, to handle a moving span without a man upon it to manipulate the machinery and to act quickly in emergencies. This question of single and double operation arises also in the case of double-leaf-bascule bridges. The double, rotating, cantilever draw consists of two swing spans, differing but little from those of the ordinary type, each resting upon a pivot pier and meeting at mid-channel, where they are (or should be) locked together so as to make the adjoining ends deflect equally and simultaneously. The other end of each swing span is locked to the masonry of the outer rest-pier, which has to act as an anchorage for the cantilevered live load.

Fig. 28*a* shows a layout of the Cleveland bridge referred to above. It is described quite fully in *Engineering News*, Vol. XXXIV, page 83.

It is not absolutely necessary to make the shore arms of the same length as the channel arms, because each or either swing may be a bobtailed draw. In that case not only should the short end be counterweighted but also there should be added near its extremity a vertical surface of sufficient area to equalize the moments of the wind pressures on the two arms about the axis of rotation. The locking gear at the meeting ends of the two swings is an awkward and unsatisfactory detail to design. The most perfect device for this connection must, of necessity, be more or less loose and clumsy; and it is likely to give trouble in operation. While this type of bridge cannot be as rigid as the ordinary swing span type, nevertheless it is possible to make it fairly satisfactory and effective. It is not probable that many such structures will ever be built, because there are other and better types of movable spans, some one or more of which will meet any special conditions better and more economically than the double, rotating, cantilever draw.

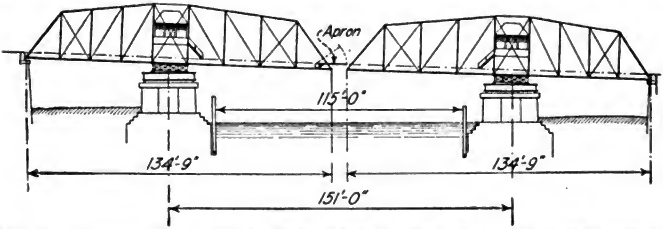


FIG. 28*a*. Double, Rotating, Cantilever Draw Bridge over the Cuyahoga River at Cleveland, Ohio.

The pull-back draw is also a very unusual type, and will always be so, for the reason that the first cost is great and its operation is expensive. This type may be divided into two classes: first, structures with one span over the entire opening, and, second, structures with two spans over the entire opening meeting at midchannel, as in the case of the double, rotating, cantilever draw. The first class requires a truss-bridge nearly, if not quite, twice as long as the width of channel between pier centres, the bottom chords thereof running on two groups of rollers that travel just half as fast as the bridge when the span is moved longitudinally. Although the shore arm may be made shorter than the channel arm, still its weight must be such that its moment will be somewhat greater than the tipping moment of the weight of the channel arm just as it leaves the farther pier. A disappearing platform will be required so as to leave space on the approach for the shore arm to move back, or else the whole bridge will have to be rotated slightly about a horizontal axis so that it can roll up onto the approach. Either method is very clumsy, and the operation of the bridge consequently must be slow. The double pull-back draw is similar to the single pull-back draw just described, except that the far end of each span has to be anchored down to a mass of masonry when the bridge is closed and ready for traffic, and the ends meeting at mid-channel must be locked together as in the case of the double, rotating, cantilever draw.

In the competition for designs for a movable structure to cross the entrance channel to the harbor of safety at Duluth, Minn., held about a quarter of a century ago, the author prepared and submitted a plan for a double, pull-back draw-bridge; and although he evolved a structure that would have worked, he was far from satisfied with the design, and in consequence, submitted another for a vertical lift—the first bridge of that type ever proposed in America for the passing of high-masted vessels.

In connection with this competition there was an amusing occurrence that illustrates the general futility of having engineers compete on plans. Designs had been called for on the basis of using the pull-back draw, the prize being one thousand dollars in cash and the engineering of the structure. The author's design for a vertical-lift bridge was selected by a Committee of five of Duluth's leading citizens as the best and most satisfactory of all the plans submitted, and it was decided to build upon it, provided the consent of the War Department could be secured. The chairman of the committee was a Norwegian, and a Norwegian engineer had submitted a design for a single-leaf, pull-back draw of mammoth proportions and a monstrosity in more ways than one—for instance, the railroad trains had to enter the structure on a twenty-five degree curve through one of the panels of one truss! Just think of steam railroad trains on such a sharp curve and what an invitation for derailment and disaster such an arrangement would be! A derailment there, even if it did not destroy the bridge entirely, would block the channel completely against navigation until the wreck could be removed and the structure repaired. The committee was firmly in favor of the vertical-lift design, but the president was insistent that his countryman receive the one thousand dollar prize, which he did by a compromise—the author being retained for a two thousand dollar fee to prepare preliminary plans for a vertical lift to submit to the War Department, with a promise of the engineering on the usual percentage basis for compensation if the application proved successful. The excuse given for not awarding the cash prize to the designer of the vertical lift was that "it pulled up and not back." In *Engineering News*, Vol. 27, page 168, and Vol. 28, page 390, will be found descriptions and estimates of cost for the various designs submitted in this competition. The outcome of the whole affair was that a special committee of U. S. Army engineers decided against permitting any bridge to be built across the entrance to the harbor of safety, toning down their adverse decision by terms of eulogy for the vertical-lift design. Years afterwards, however, the War Department permitted the building at the crossing of an aerial ferry, as being less dangerous for navigation than any other type of bridge.

There is described in the *Engineering Record* of July 31, 1897, a double pull-back draw over the River Dee at Queensferry, Scotland. It provides a clear opening of one hundred and twenty feet, and cost about $70,000. It is of the telescopic type, *i. e.*, each half of the opening span pulls back and telescopes into the approach span. This bridge must certainly be lacking in rigidity, and the transferrence of the wind loads to the piers can only be done by transverse bending of the truss posts of the approach spans, as the passage of the movable arm through its interior effectively precludes any attempt to provide vertical sway bracing except at the far end. If the vertical posts are properly figured to carry transversely the excessive wind load required by the British standard regulations, their sections must be enormous. Because of its inferiority to several other types of movable bridges, it is more than likely that no more structures of the pull-back type will ever be constructed.

Trunnion-bascule and rolling-lift bridges are treated at length in Chapter XXX, which deals with "Bascule Bridges."

Jack-knife or folding bridges were a freak design that passed out of existence more than a decade ago. Two of them were built in Chicago, but they proved to be so light and vibratory and were so continually out of order that they were soon removed. Each half of a jack-knife bridge consists of two steel towers, from the top of which are suspended by tie-rods the two leaves of the floor. These are hinged together at their point of junction, and when the draw is to be opened this point rises, the other ends of the leaves move downward, and each half of the floor assumes the position of an inverted V. In this position a portion of the space between the piers is left free for the passage of vessels; and it was claimed that "the raised floors form effective guard gates." Unfortunately, though, the said guards are badly placed, as there is left in front of each of them a big opening in the floor for animals and vehicles to fall into.

Concerning this type of structure in 1897 the author wrote thus in his *De Pontibus:*

> "The jack-knife or folding bridge is a type of structure which is not at all likely to become common. There have been only two or three of them built thus far, and they have been often out of order; moreover, considering the size and weight of bridge, the machinery used is powerful and expensive. The load on the machinery while either opening or closing the bridge is far from uniform, and the structure at times almost seems to groan from the hard labor. The characteristic feature of the jack-knife bridge is the folding of the two bascule leaves at mid-length of same when the bridge is opened. The loose-jointedness involved by this detail is by no means conducive to rigidity, nevertheless these structures are stiffer than one would suppose from an examination of the drawings. The Canal Street Bridge, Chicago, is of this type; and its design is illustrated in *Engineering News* of December 14, 1893."

Anyone desirous of learning more concerning this defunct type of movable bridge is referred to *Engineering News*, Vol. 25, page 486, and Vol. 30, page 480.

Vertical lift bridges are treated at length in Chapter XXXI.

The gyratory lift bridge is another freak structure—impracticable, uneconomic, but exceedingly ingenious. The design was evolved and patented by Eric Swensson, Esq., C. E., of Minneapolis, for a crossing of "The Narrows" on Lake Minnetonka near that city. As far as the author knows, the proposed structure was never built. It was described in *Engineering News*, Vol. 59, page 367. It consists of a pony-truss or plate-girder span suspended by trussed hangers from trunnions bearing on a tower at each abutment. The draw is opened by revolving the main roadway trusses in an arc around the horizontal longitudinal axis marked by the trunnions. The upper portions of the trussed hangers carry counterweights equal in weight to the suspended span so that friction and wind are the only forces for the machinery to act against. Motors and gears are placed in the towers, the said gears engaging with circular racks attached to the hangers and extending over arcs of 180 degrees, so as to control directly the turning of the span. When the bridge is in its normal position, wedges are to be placed under the extreme ends so that the live load will be carried directly to the abutments and not through the hangers to the trunnions. Solely on account of its novelty and the ingenuity employed in its evolution, the illustration given in *Engineering News* is reproduced in Fig. 28*b*. This type of movable bridge is uneconomic in the extreme, the two most expensive features being the excessive length of the moving span, as compared with the horizontal clearance, and the two sets of operating machinery. If one will compare this structure with the vertical lift adopted as standard by the Southern Pacific Railway Company and illustrated in Figs. 31*h* and 31*i*, he cannot help being struck by the great difference in the economics of the two types. Another objectionable feature of the "gyratory lift" is the turning of the floor bottom upwards. This would preclude the employment of any kind of pavement, and would necessitate, for a highway bridge, the adoption of a plank floor—a detail that is incompatible with first-class bridge construction.

The aerial ferry, transporter bridge, or *transbordeur*, is a type of construction which may very properly be termed a cross between a bridge and a ferryboat. From the point of view of efficiency in transportation it is decidedly inferior to the former but somewhat superior to the latter.

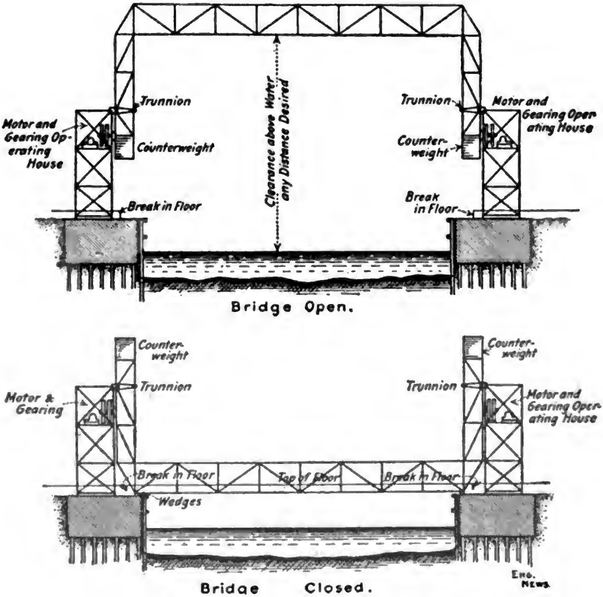


FIG. 28*b*. The Gyratory Lift Bridge.

Its excuse for existence is solely that the navigation interests will consent to its being built in certain localities where a real bridge of any kind, except one of long span and great vertical clearance, would not be permitted. It consists of two towers, an overhead span high enough to clear the masts of the tallest vessels, a track on the span, a car running upon the track, and a travelling platform suspended from the car. There is but one structure of this type in America, but a number of them have been built in Europe. The American one, as already mentioned in this chapter, crosses the entrance channel to the harbor of safety at Duluth, Minn. A description of it is given in *Engineering News*, Vol. 47, page 227. It consists of a riveted truss span of 394 feet supported on steel towers resting on pile and concrete foundations, and having the bottom chord of the span 135 feet in the clear above high water. The ferry-car is suspended by stiff riveted hangers from trucks running on tracks placed within the bottom chords of the trusses. It is proportioned to carry a loaded street car weighing 21 tons and a live load of 100 pounds per square foot on all the floor space not occupied by the car. In Fig. 28c is given a sketch of this bridge. It will be noticed that the ferry-car has to cross not only the canal but also a driveway on each side thereof. At each end of its travel it passes inside of the tower out of the way of everything and connects to a short ramp leading to the street. The ferry-car is operated by electricity at an ordinary speed of four miles per hour with capacity for moving much more rapidly should the necessity arise.

The first *transbordeur* built in Europe is the one at Rouen, illustrated in Fig. 28*d*. It was designed by a French engineer, Monsieur F. Arnodin, and a Spanish architect, Señor A. de Palacio. As the illustration indicates, the overhead structure is of the suspension type, which, owing to the long span and light live load that generally are ruling factors in aerial ferry structures, is eminently fitted for the purpose.

Other bridges of this type are as follows:

Transbordeur across the harbor at Marseilles, France, with a span of 541 feet and a car 33 feet by 39 feet. (See the *Proceedings of the Institution of Civil Engineers*, Vol. 167, p. 404.)

Transporter Bridge at Newport, England, with a span of 645 feet and a vertical clearance of 177 feet. (See same publication, page 405.)

The Widner and Runcorn Transporter Bridge over the Mersey, England, with a span of 1,000 feet (see *Proceedings of the Institution of Civil Engineers*, Vol. 165, p. 87).

Cableway at Brighton, England, with a suspension span of 650 feet and a car capable of carrying only eight persons. (See *Engineering News*, Vol. 33, p. 67.)

Transbordeur near Bilboa, Spain, with a span of 500 feet and a car capable of carrying 150 passengers, the time for transit being one minute. (See *Engineering News*, Vol. 30, p. 260.)

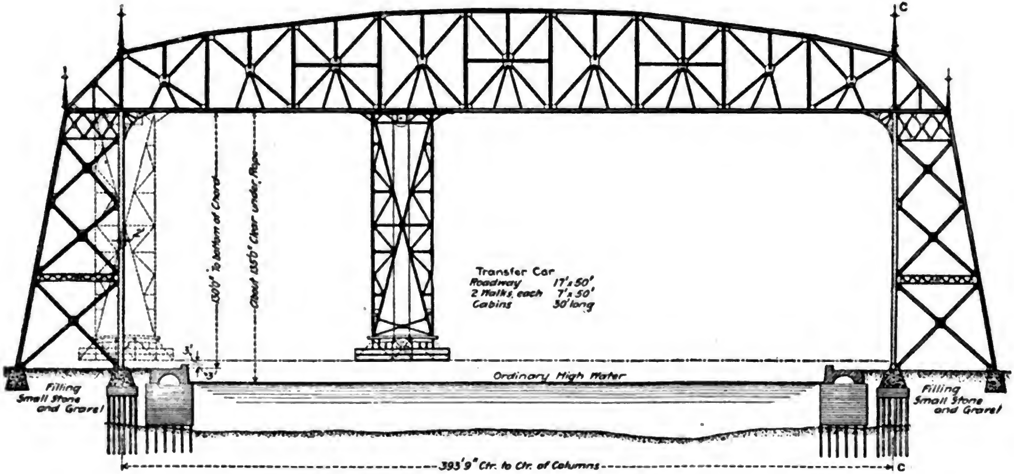
Transbordeur at Bizerte over the Canal (Tunis) with a span of 358 feet and a car 30 feet by 25 feet. (See *Le Génie Civil* of Nov. 21, 1903.)

Transbordeur over the Loire at Nantes, France, with a cantilever suspension span of 490 feet and a car 40 feet by 33 feet. (See the *Railroad Gazette* of August 26, 1904.)

Transporter Bridge across the Manchester Ship Canal, having a clear span of 1,000 feet. (See the *Scientific American*, of May 28, 1904, p. 420.)

Transbordeur at Martron, France, having a span of 460 feet and a car 46 feet by 38 feet. (See *Le Génie Civil* of Nov. 21, 1903, p. 35.)

The advantages usually claimed for the transporter bridge are as follows: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Transfer Car
Roadway 17'x50'
2 Walks, each 7'x50'
Cabins 30' long
130'0" to bottom of Chord
About 135'0" Clear under Rope
3'
1'3"
Ordinary High Water
Filling Small Stone and Gravel
Filling Small Stone and Gravel
393'9" Ctr. to Ctr. of Columns
C

FIG. 28c. Transporter Bridge at Duluth, Minn.

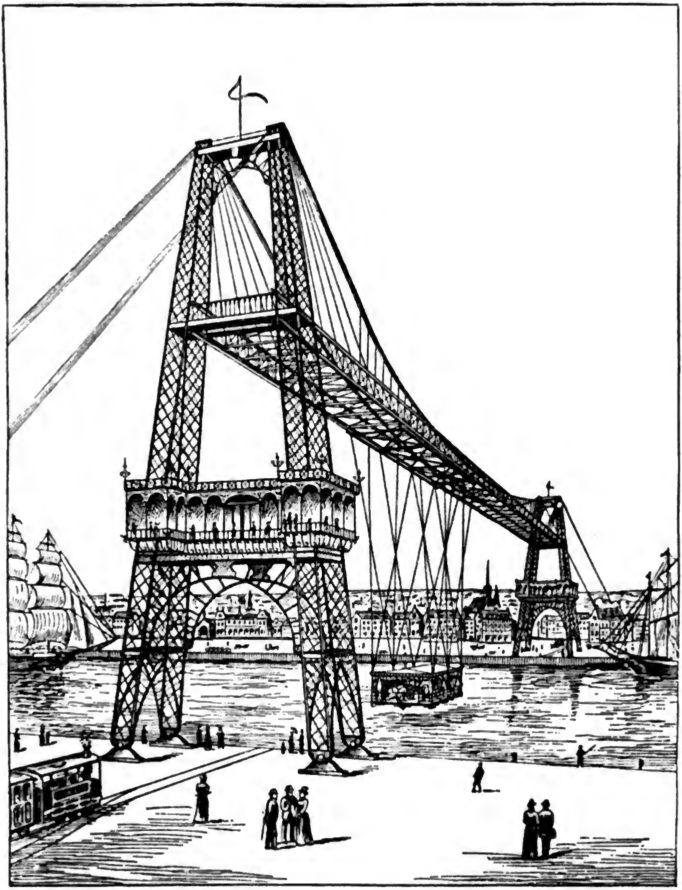


FIG. 28*d*. Transporter Bridge at Rouen, France.

1. The channel to be crossed is left entirely clear at all hours, without requiring vessels to make any special signals or modify their rate of speed any more than they would in case of a cross-channel ferry.
2. No increase of distance or ascent or descent is forced on the traffic in order to cross from one shore to the other.

The disadvantages are these:

1. The limited carrying capacity of the structure.
2. The long time usually required to cross.

If the author were ever called upon to design a transporter bridge, he would effect a great improvement by widening the structure so as to provide for a double track and would carry on it four or more cars. These cars would always travel upon the right hand track, and would run onto a single track at each end of span where they would discharge and take on passengers. Again, he would use powerful electric motors so as to travel at high speed. By these means, the carrying capacity of the bridge would be multiplied many fold and the time required for transit would be reduced to a minimum, because the intervals between cars could readily be made as small as one minute, requiring only sufficient time to unload and reload the foot passengers and vehicles. The car should be made double deck, the pedestrians being carried above; and the roadway should have a double track, the right one being for the use of a single street-car and the left for two, or possibly three, wagons. At the end of the trip the car would leave first, and the wagons would follow immediately, edging over to the right so as to permit of the ingress of the oncoming car, which in its turn would be followed by wagons to occupy the left hand side. While the vehicles would be going off and others getting on, the upper deck could easily be emptied of its pedestrians and refilled.

There is prevalent an idea that floating or pontoon bridges are employed only in communities where the inhabitants are absolutely too impecunious to build a permanent structure, and that they are a makeshift in every sense of the word, are expensive to operate, and require much time to open and close for the passage of vessels. This popular notion is not altogether correct, for there are some locations in which floating bridges with movable spans are not only legitimate construction but are truly economic. For instance, up to 1908 (and possibly to the present time) the Chicago, Milwaukee & St. Paul Railway Company was operating four pontoon draw spans, two of them having been in use since 1875 and one since 1883. They appear to have given satisfactory service and to have proved economical. Their life has been about twelve years and their first cost about twenty or twenty-five per cent of that of a permanent structure. Concerning these C., M. & St. P. R'y bridges, the reader is referred to *Engineering News*, Vol. 59, p. 474. It is quite true that floating draws are usually expensive both to operate and to maintain, notwithstanding the experience of the C., M. & St. P. R'y Co. to the contrary. The governing conditions must have been more favorable than those often encountered; for the author knows of a number of pontoon bridges that had to be abandoned soon after their completion because of excessive difficulty and expense in maintenance. Pontoon bridges have been built from time immemorial and are still much used for military purposes. It is practicable to employ a floating draw with fixed approaches of either spans or trestle; but, on account of the variation in water level, this expedient is resorted to generally only for temporary purposes. Pontoon bridges are often short lived, as the boats are occasionally carried far down stream during freshets, or are broken up by floating trees, logs, or vessels. Under adverse conditions they are very perishable and are easily put out of commission. There is one type of pontoon, though, that is a necessity in certain places, viz., those located at the ends of ferries where there are large variations in the water level. Such structures, strictly speaking, may not really come within the scope of this chapter. In *Engineering News*, Vol. 21, page 308, there is given a description with working drawings of a passenger ferry bridge for the New York, Lake Erie & Western Railroad; and in *The Engineering Record*, Vol. 48, p. 489, there will be found a very complete article treating of the New Orleans Railway Incline Bridge.

In *The Engineer* of June 28, 1912, there is described a novel and unique design for a floating bridge across the Hoogly River between Calcutta and Howrah, India—in fact, there are two designs, quite similar in character, as illustrated in Figs. 28*e* and 28*f*. The banks of the river are of mud, and the bed is silt so loose as to be incapable of supporting with safety any load whatsoever. On this account a floating bridge is a necessity. Messrs. Head, Wrightson & Co., the designers, have solved the problem in a masterly and clever manner. Each of their layouts consists of two approach spans of about 480 feet each and two bob-tailed swing spans having the longer arms 150 feet in length and the shorter ones about 70 feet. At the shore ends the approach spans rest on masonry abutments, and at the river ends both they and the swing spans are supported by immense pontoons, each composed of eight water-tight steel cylinders 15.5 feet in diameter and 220 feet long. In one design this platform of cylinders floats on the surface of the river, but in the other it is submerged, being firmly anchored into the mud by vertical rods attached to steel cylinders filled with concrete and buried therein. In both cases the pontoons are anchored up and down stream by chains attached to similar buried cylinders situated some 400 feet above and below the bridge tangent. If the pontoons float upon the surface of the river, the outer ends of the shore spans rise and fall with the changes in water level, thus putting heavy grades in the track; but if they are submerged, the rising and falling of the water will have no appreciable effect on the superstructure. Provision was made for repairing or removing, one at a time, the various cylinders without interfering with either the traffic over the

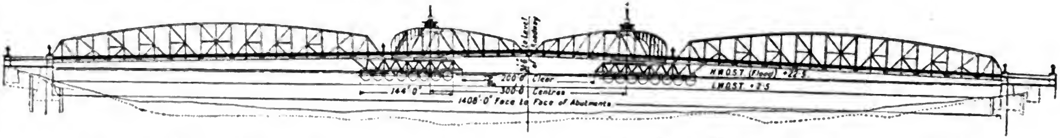


FIG. 28e. Proposed Pontoon Bridge over the Hoogly River between Howrah and Calcutta, India. Design No. 1.

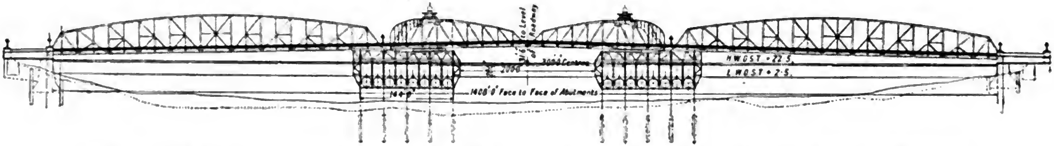


FIG. 28f. Proposed Pontoon Bridge over the Hoogly River between Howrah and Calcutta, India. Design No. 2.

bridge or the operation of the swing spans. For further details of these two designs the reader is referred to the columns of *The Engineer*.

In *Engineering News*, Vol. 70, p. 1,018, there is given a most interesting description of the new pontoon bridge over the Golden Horn at Constantinople. It has a clear roadway of 46 feet and two 18-foot sidewalks, the total length between abutments being 1,530 feet. The swing span, which is hinged at one end, can turn through 180 degrees. It is moved by propellers fore and aft, operated electrically. In the flanking portions of the bridge the pontoons are placed longitudinally, one on either side of the structure; and they carry trusses spanning transversely of the bridge, on which trusses the floor rests. The movable or draw section of the structure has transverse pontoons which leave, when it is closed, two boatways, each 39 feet wide with 17.5 feet of vertical clearance. The draw has a length of 205 feet.

To protect the structure-traffic when the span is open, gates should be provided at the ends of all movable spans of highway bridges. They should be arranged so as not to interfere with the traffic when the span is closed and so as completely to block all passage before it begins to open. Where the traffic in both directions uses the same roadway, the gates are best arranged in pairs at each end so that one of each pair can be closed to oncoming vehicles and pedestrians and the others shut just as soon as the traffic is off of the movable span. Where separate roadways are employed for the travel in the two directions, a single gate at each end of each roadway is generally used. These are arranged so that the gates obstructing the oncoming traffic are shut first: and the other gates are closed against pedestrians as soon as the movable span is cleared. This same construction is adopted also for single roadways with light traffic in both directions that can be easily handled. In such cases it is not infrequent to omit the gates altogether, the bridge tender merely stretching a heavy rope across the roadway at each end. Various types of gates are employed, those swinging in a horizontal plane being the most common. The pivots are placed near the trusses; and when not in use, the gates are swung up against the latter out of the way of passing vehicles, latching into place automatically. Various types of folding gates that are operated vertically about horizontal axes have been used, the main objection to them being the danger of striking passing vehicles and pedestrians when they are being lowered. This, of course, is not very important where gatemen are employed; but when the gates are operated from the machinery house, it is a serious matter. The same is true also of lifting gates which are dropped across the roadway from above or raised from beneath the floor.

The character of the construction of the gates will depend on the use to which they are to be put. If they are merely to serve the purpose of a tell-tale, a very heavy gate is not needed; but where provision is to be made for collision with a horse, wagon, or automobile, a substantially designed construction is necessary. Stops should be provided at the ends of swinging gates to hold them when shut. These may consist of pointed rods pivoted above the bottom of the gate and stuck into the pavement when the said gate is closed. The gates are usually made of structural shapes, although wood is sometimes employed. The horizontally-swinging and the direct-lifting gates are generally of latticed construction, while the folding gate is made up of angle flanges at top and bottom connected by hinged parallel bars. The operation may be effected by hand or by machinery controlled by the bridge operator. As a rule, gatemen are employed to handle the traffic and to operate the gates as well. This is the surest method of preventing accidents. With very heavy traffic the operation of the movable span and that of the gates should be done by different men, working with an efficient system of signals.

The question of what is the best kind of power for operating movable bridges is not difficult to answer, for where electricity is available it is the best and usually the cheapest energy to employ. But there are movable bridge locations where electricity is not available, and in such cases the best power to adopt is that produced by a gasoline engine. The latter is superior to the steam engine, because with steam the fires must always be kept going at great expense for both fuel and attendance, but with a gasoline engine, except when the river traffic is very dense, causing constantly-recurring calls for an open draw, there is no burning of fuel except during operation. Steam machinery used to be employed quite generally for operating swing spans, but no one nowadays ever thinks of adopting it. Hydraulic power has also been used in the past for operating bridges, especially in Europe; but it unavoidably involved the employment of such excessively expensive machinery that it never became popular. Compressed air has been adopted a few times in both America and Europe for operating swing spans, not, however, as a primary but as a secondary power. It is not likely ever to be used in the former manner, because the existence of an independent source of supply of compressed air in the vicinity of a movable bridge involves a most improbable combination of conditions; hence it would be necessary to compress and store the air by an electric motor, gas engine, or, possibly hydraulic machinery in case there was an available water power in the neighborhood. Electricity is certainly the ideal power for handling movable spans, especially when there is available more than one source of supply. If there be but one, and if the stream carry much traffic, as a matter of precaution the designer should install either a storage battery or an auxiliary gasoline engine capable of operating the structure at moderate speed.

Where natural gas is available and cheap, it is sometimes economic to adopt a gas engine; but even under these conditions it is difficult to compete successfully with electricity, especially when the items of interest, depreciation, and repairs are duly considered, for these are much greater when either gas or gasoline is used than when electricity is the motive power. The much greater weight and heavier vibration of the gas engine or the gasoline engine as compared with the electric motor militates materially against its employment for operating bridges, because it costs money to support weight even in the tower of a swing span, and excessive vibration is certainly a disadvantage that should not be ignored.

In *Engineering News* of Oct. 13, 1910, there is a paper by S. F. Nichols, Esq., E. E., who is an acknowledged authority on electrical engineering, entitled "The Electrical Operation of Drawbridges." His statements concerning the superiority of electricity as the motive power are so clear and conclusive that the author takes the liberty of quoting from the said paper as follows:

> "The electric motor has many points to recommend it, with few disadvantages. It is very light and compact, and it is very conveniently reversed. It is capable of sustaining a very heavy overload for short periods, which enables it to take care of the very difficult problem of accelerating a heavy mass and also of operating the bridge against high wind pressures that may occasionally be experienced. It is almost noiseless in operation. It requires comparatively little attention, and when periodically inspected the possibility of its getting out of order and refusing to work is very remote.
>
> "Being compact, it can be located very close to the point where the power must be used. This makes it possible to locate the operator at the most convenient position from the standpoint of accessibility or where the best view can be obtained of the river or railroad or highway traffic. The motors can be located on a moving portion of the structure while the operator's house is located on the fixed part."

Mr. Nichols' statement regarding the overload capacity of a motor applies, of course, to direct-current motors only; but the other points are true for alternating-current motors as well.

For bridges of importance it is certainly good practice to have as many sources of power as possible available for use in an emergency, and if both electric current and an independent source of energy can be furnished, the duplication is well worth the extra cost that it involves; for reliability is the prime *desideratum* to be attained.

In respect to the amount of power required for mechanically operated bridges, there is a good rule given by Albert Henry Smith, Esq., C. E., in his discussion of Schneider's paper on "Movable Bridges," viz., to allow one horse-power for each fifteen tons of weight to be swung. This provides ample margin for taking care of excessive wind pressures, and gives plenty of power to open and close the bridge rapidly. The rule was established for swing spans; and it applies very well on the average for vertical-lift bridges, *provided that the figured tonnage includes the counterweights and all moving parts.*

No matter what kind of power may be employed, every movable span should be provided with a means for operating it slowly by hand in order to meet the possible emergency of the failure of all other powers.

It is often necessary for an engineer to make a rough or hurried estimate of cost of power installation for a movable span. Of course, this will vary greatly with the numerous ruling conditions, especially the time of operation; but assuming them all to be averages of those usually encountered, the author has prepared the diagrams recorded in Fig. 28*g*, which show for swings, vertical lifts, and bascules the first costs per weight unit of 100,000 pounds for power installation. These curves, which are applicable to direct-current and gasoline engine but not to alternating-current operation, are recommended for use in preliminary estimates only, as each individual case should sooner or later be worked out accurately after all the conditions have been thoroughly and finally determined. With alternating current the cost of power installation will be about twenty-five per cent greater than that for direct current.

The best kind of movable bridge to adopt for any crossing will depend greatly upon the conditions that exist there. Generally the vertical lift or the bascule is superior to the swing for the following reasons:

First. Either of the lifts provides one comparatively large clear opening instead of the two smaller ones involved by the swing.

Second. It offers less obstruction to the flow of water, owing to the absence of the draw rest and (generally) also to the smaller number of piers.

Third. The cost of maintenance is less because of the necessity of maintaining a perishable draw rest for a swing span.

Fourth. The danger of the span's being struck by passing vessels is much greater in the case of a swing than in that of either kind of lift.

Fifth. The time of operation is two or three times as long for a swing as for either style of lift.

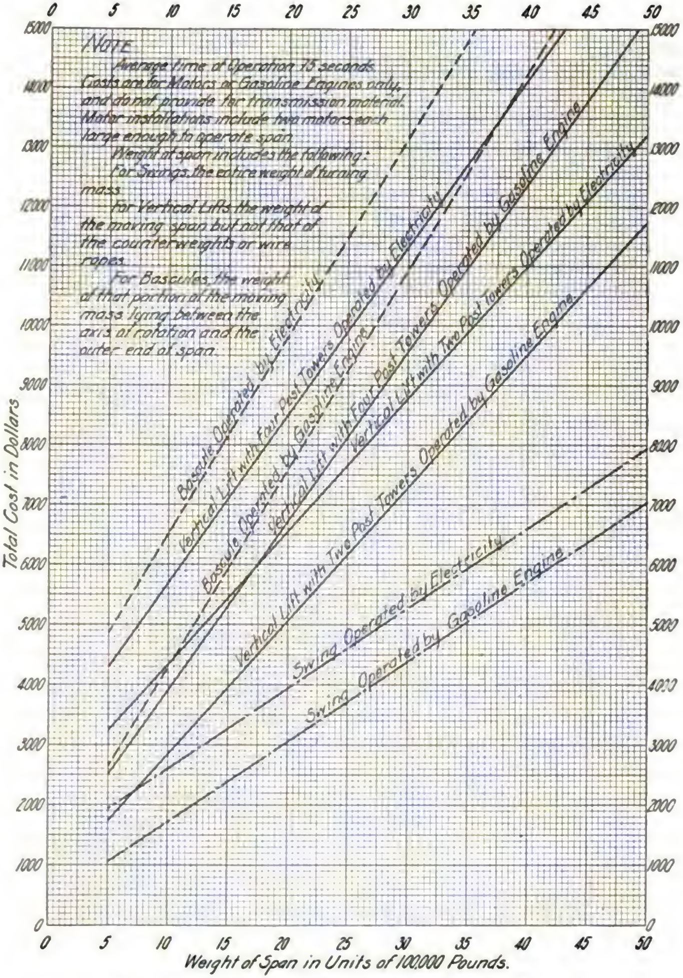
Sixth. The lifts generally afford better automatic adjustment of the railroad tracks than do swing spans, although with proper precautions there should be no danger of accident on account of derailment caused by improper track adjustment. A serious accident from this cause occurred at Atlantic City on October 21, 1906, in which a number of people were killed.

Seventh. The swing bridge often interferes with adjacent valuable property, which neither type of lift does, because the location of lifts is always confined to the city street or the company's right-of-way.

Eighth. In case of future enlargement of bridge to accommodate an increase of traffic, the swing bridge has to be torn down and rebuilt, while a vertical lift or a bascule can simply be duplicated alongside.

Ninth. The wider the roadway of a swing bridge the more obstructive does it become to navigation, while the widening of a lift does no harm thereto whatsoever.

Tenth. In passing vessels with low masts a swing has to open just as fully as for a high-masted craft, which is not the case with a vertical lift or a bascule. In this regard the vertical lift has a decided advantage over the latter in that the deck remains horizontal.



NOTE

Average time of Operation 75 seconds. Costs are for Motors or Gasoline Engines only, and do not provide for transmission material. Motor installations include two motors each large enough to operate span.

Weight of span includes the following:

For Swings the entire weight of turning mass.

For Vertical Lifts the weight of the moving span but not that of the counterweights or wire ropes.

For Bascules the weight of that portion of the moving mass lying between the axis of rotation and the outer end of span.

FIG. 28*g*. Cost of Power Equipment for Movable Spans.

Eleventh. In sand-bearing streams the protection works for the pivot pier cause a deposit of sediment and thus often tend to obstruct navigation.

Twelfth. Generally speaking, the first cost of a swing bridge is more than that of either the vertical lift or the bascule, although there are occasional exceptions. This question is discussed at length in Chapter LIII.

Comparing the vertical lift with the bascule, the former has a few advantages, among which may be mentioned the following:

First. The floor is always horizontal, permitting the use of a block pavement, which cannot well be employed on a bascule.

Second. Great wind pressure during operation has no appreciable effect on a vertical lift, while it may cause serious delay to a bascule, or even, under extreme conditions, prevent its operation altogether.

Third. As previously indicated, the vertical lift does not have to rise so high for low-masted passing craft as does the bascule, and thus it saves a small amount of time.

Fourth. In railroad bridges when the moving span is down it acts just like any fixed span as far as operation under traffic is concerned, which cannot be said for either the bascule or the swing; or in other words, for railroad traffic the vertical lift is the most rigid of the three types.

Fifth. In case of a shifting channel, it is feasible at very moderate expense to make a number of the spans alike and to arrange, for any time in the future, to have the towers and machinery taken down, transferred, and re-erected so as to lift any one of the said equal spans. This could not by any possibility be done in the case of any other type of movable structure.

Sixth. The vertical lift, when its towers do not rest on flanking spans, lends itself readily to a future raising or lowering of the grade line in a way that no other type of movable span can possibly do; for all that is necessary is to change the elevation of its bearings at the four corners and to modify slightly the transverse bracing of the towers. If a change of grade be anticipated when the plans are being prepared, provision should be made therefor by increasing adequately the heights of the towers; but if at any time the grade on a vertical-lift bridge of the type mentioned, for which no such preparation had been made, has to be raised to such an extent that the required clear headway will be interfered with because of the counterweights reaching the new decks of the approaches in the towers, the result desired can be accomplished by arranging for a small portion of the said approaches to move either laterally or vertically out of the way of the counterweights whenever a very-tall-masted vessel has to pass. For any other vessel, however, these moving approaches would not have to be utilized; consequently, they would very seldom need to be operated.

Seventh. As explained in detail in Chapter LIII, for large bridges, long opening spans, and expensive substructures the vertical-lift bridge costs less than the bascule.

In the 1908 *Transactions of the American Society of Civil Engineers* there is a masterly paper upon the subject of "Movable Bridges," by Past-President C. C. Schneider; and no one who aims to be a bridge engineer can afford to neglect studying it thoroughly.

CHAPTER XXIX

SWING BRIDGES

THERE are three general classes of revolving draw bridges, viz., the rim-bearing, the centre-bearing, and the combined rim-bearing and centre-bearing. In each of these classes a bridge may be either equal armed or bob-tailed. Again, rim-bearing draws may have either one or two supporting points per truss at the pivot pier. Centre-bearing draws are generally arranged so as to carry the dead load on the pivot and the live load on either a drum or four carriages formed of groups of rollers. Combined rim- and centre-bearing draws carry a portion of the dead load on the pivot and the remainder on a drum, with the live load supported as in the last described case. Swing bridges may be classified also as to the character of their main girders thus—plate girder swings; open-webbed, riveted-girder or riveted-truss swings; and pin-connected-truss swings. Another general division of swing bridges is in relation to their continuity or the reverse for the travel of live load shear across the pivot pier. Most structures are more or less continuous in this regard; but a few have been designed and built in such a manner that, when the ends of the arms are raised to their normal position for the span closed, the two halves of the structure are entirely independent of each other in respect to all kinds of loading and all conditions thereof. Many years ago the author designed a small swing span of this kind. It was operated by man power, and required so much time and energy to lift the ends adequately that he has never repeated the experiment. Other engineers have tried similar designs, and, undoubtedly, with equally unsatisfactory results, because the type has not persisted. Once more, swing spans may be divided into through, half-through, and deck.

In addition to these various classes of swing spans one will occasionally run across some freak design that, if perpetuated, would form a class of its own; but such abnormal variations from general practice are either still-born or very short lived. One of the most glaring cases of this kind is described in *Engineering News*, Vol. 29, p. 141. Its characteristic feature is the floating of the movable span by means of a water-tight steel tank working loosely in a similar shell encased by the concrete of the pivot pier, the space between the two shells being filled with water. Setting aside the abnormally high cost of such a contrivance, just think of what would happen should the inner shell spring a leak or should the water freeze! It was suggested by someone to use mercury instead of water, but that would prove a costly expedient, for mercury is expensive, and it has a bad habit of evaporating when exposed to the air. This freak design won the first prize in a competition on plans at Sydney, N. S. W., in 1892, but the author has never heard that the bridge was constructed according to the said design. It is but fair to state that at Sydney there would be no danger of the water freezing. It is stupid awards of this kind that discourage expert bridge engineers from competing on plans.

With all the preceding groupings it is evident that there are many possible kinds of swing spans differing from each other quite materially. The most common kinds are the rim-bearings ones, and of these the deck-plate-girder and the riveted-through-truss types are the most numerous. Half-through, plate-girder swings occur occasionally, and in times past pin-connected rotating draws were very common; but of late, as explained at length in Chapter XXXII, riveted trusses have supplanted pin-connected ones in spans of ordinary length, and, indeed, often in quite long spans; and this statement holds good for movable as well as for fixed bridges.

The choice between rim-bearing and centre-bearing swings is almost entirely a matter of taste; for there is no great difference between them in the cost, what little there is being in favor of the latter. In general it may be stated that while the rim-bearing draws are often more rigid and stable, the centre-bearing draws move with less friction. In respect to the minimum dimensions for pivot piers, the centre-bearing structure has somewhat the advantage; but with the type of rim-bearing draw that the author has for many years been building, in which the diameter of the drum is equal to the perpendicular distance between central planes of trusses and in which there are provided at least eight points of bearing for the span upon the drum, the saving in substructure cost by adopting the centre-bearing type is not great. Since writing *De Pontibus*, the author has had occasion to modify his opinion concerning the comparative merits of rim-bearing and centre-bearing swings, because the latter type has been so materially improved in the last two decades that it has today a slight advantage in both initial cost and ease of operation over the former; but he still adheres to the adverse opinion expressed in that treatise concerning swings that divide the load between rim and pivot. While it is not impossible to build satisfactory bridges of that type, there is always a certain amount of ambiguity in regard to the division of the load between those places. The choice between rim-bearing and centre-bearing swings will often depend upon the character of the pivot pier. In a late alternative design for the moving span of the Pacific Highway Bridge over the Columbia River near Portland, Oregon, which is being engineered by the author's firm, the swing span was made rim-bearing, and the pivot pier was a six-foot-thick shell of concrete covered by a reinforced concrete cap four feet thick, the foundation for the pier being very long piles. This construction was estimated to cost somewhat less than a pivot-bearing swing supported by a solid pier. That there are still conflicting opinions among high authorities concerning the relative merits of the rim-bearing and the centre-bearing swing can be ascertained by comparing the opinions of C. C. Schneider, Esq., Consulting Bridge Engineer, as expressed in his paper on "Movable Bridges" presented at the April 3, 1907, meeting of the American Society of Civil Engineers, and of C. H. Cartlidge, Esq., Bridge Engineer of the Chicago, Burlington, and Quincy Railway, as stated in his paper, "The Design of Swing Bridges from a Maintenance Standpoint" presented at the April 18, 1906, meeting of the Western Society of Engineers.

Mr. Schneider says: "The centre-bearing type, designed in accordance with good modern practice, offers more advantages than the rim-bearing type, and should always receive the first consideration in determining upon a design. It requires less power to turn, has a smaller number of moving parts, is less expensive to construct and maintain, requires less accurate construction than the rim-bearing bridge, and does not as easily get out of order. The structural and the operating or machinery parts are entirely separate, and when the bridge is closed it forms either two independent fixed spans, or a fixed span, continuous over two openings, resting on firm, substantial supports. There are no ambiguities in the calculations in reference to the distribution of the load, and the distance required from base of rail to masonry is generally less than that required for a rim-bearing bridge with proper distribution of the load over the drum. Any irregular settlement of the masonry does not materially affect its operation.

"On the other hand, the rim-bearing bridge requires a circular girder or drum of difficult and expensive construction, a ring of accurately-turned rollers, and circular tracks, which require great care in their construction and delicate adjustment in their erection in order to make the bridge operate satisfactorily. Repairs are troublesome and expensive, and any irregular settlement of the masonry will throw the whole turning apparatus out of order."

On the other hand, Mr. Cartlidge says: "The writer's experience with centre-bearing draw-spans has been such as to prejudice him against them for spans of any magnitude. It seems difficult at any reasonable cost to proportion the pivot-bearing so that it will not wear; and any wear on a pivot-bearing is expensive to repair. On one draw the wearing away of the bronze bearing in the pivot allowed the upper and lower castings to rub, making the turning of the draw a very noisy operation, while the few wheels provided to steady the span while turning were overloaded and cut the circular track badly."

It is a difficult matter to choose between the opposing dicta of two such eminent authorities. Mr. Schneider's experience, extending over an unusually long professional career, has been mainly in designing and manufacture, and Mr. Cartlidge's in erection and operation. Mr. Schneider is of the opinion that centre-bearing bridges are adapted for single-track structures of any span, but for four-track bridges and heavy highway bridges carrying wide city streets they are not suitable; while Mr. Cartlidge would use centre-bearing swings for short, light spans, and either rim-bearing or combined rim-and-centre bearing swings for long, heavy ones.

Mr. Cartlidge's explanation of how he divides the dead load between rim and centre shows how uncertain must be the true distribution. Referring to one of his bridges he says: "The beams bearing on the centre casting are designed to carry half the dead load to the casting. The adjustment of the load is by means of shim plates between the beams and the top of the pivot. The adjustment is made during erection, the beams first being allowed to rest on the rollers with the centre casting clear. The centre is then jacked up until the drum just clears the wheels. After noting the amount of the lift, shims to half its amount are put in and the beams lowered to permanent bearing. This is, of course, only an approximate method." It certainly is only approximate; and when the adjusting is finished, the ratios of load division will be somewhat uncertain, but nothing like as much so as later after the pivot-bearing has begun to wear, for the more it wears the greater will be the share of the load carried by the rim. The author certainly prefers either the centre-bearing or the rim-bearing swing to the hybrid design. As before stated, his practice has been mainly (and especially in the early portion of his professional career) confined to rim-bearing swings, nevertheless he has become convinced that centre-bearing ones, everything considered, are the best; for he has had troubles of his own with rollers getting out of adjustment. Mr. Cartlidge says: "One great advantage which accrues to a centre bearing is that of ease of turning; and while everything is new and in adjustment this advantage is realized. Should there be any excessive wear, however, this is soon lost, and it is necessary to make bearing areas as large as practicable, in order to prevent such wear.

"The complications involved by the use of a rim-bearing centre are more theoretical than actual, as experience with spans of widely varying length has demonstrated."

It is quite evident that both rim-bearing and centre-bearing swings have given considerable trouble in operation in times past; but the author is of the opinion that those of either type, designed and built today in strict accordance with specifications that are based upon the accumulated knowledge concerning the weak points of old structures of the said type and how to avoid them in the future, would give equally satisfactory service—but, as before indicated, this conclusion does not apply to the hybrid type.

The weak points in the rim-bearing swings were too shallow and inadequately stiffened drums; adjustable radial spider rods held to spacing by light bars or light channels; centre castings insufficiently anchored to the masonry; track segments too thin or of cast iron, or both; inadequate connections of track segments to drum or masonry or to each other; faulty contact between track segments and drum; spider rods of too small diameter; unscientific connections of brackets to drum; and improperly designed operating machinery.

The weak points in the pivot-bearing swings were centre bearings of cone-shaped rollers or balls, failure to provide proper sliding surfaces, and excessive bearing pressures.

The necessity for deep drums has already been dwelt upon, shallow ones never having been a weakness of the author's. Schneider states that the depth of the drum should be not much less than one-half, in no case less than four-tenths, of the distance between the centres of support. There should be eight supports for single-track and twelve for double-track swings. The trouble that came from adjustable spider rods does not exist in modern rim-bearing swings; for the detailing has been fundamentally changed by using a stiff ring of two channels held to gauge by batten plates passing between the wheels, with rigid radial arms riveted to it and to the pivot ring. The wheels run on short axles, adjustable radially, and have tool steel or bronze washers to prevent their being turned by friction and, consequently, put out of adjustment. Centre castings are now being made more substantial than they were formerly, and are being buried for most of their length in the concrete that forms the top of the pivot pier. Track segments, too, are being made thicker and are much better connected to each other and to the metalwork and the pier than they used to be years ago, consequently there is now more perfect contact between rollers and drum. One of the greatest improvements, though, consists in providing adequately and scientifically designed operating machinery and connecting it firmly and rigidly to the structural metalwork. In centre-bearing swings the cone-shaped rollers and the ball-bearings have been abandoned, and bearing disks of ample size and satisfactory material are being employed.

There is a feature of construction of old type drums that is deserving of passing notice on account of its glaring inefficiency and crudeness of manufacture, viz., the insertion of a so-called "rust joint," composed of iron turnings or filings and acid, between the flange of the drum and the upper track segments. It used to be made from a quarter to a half inch in thickness, and it invariably sooner or later was squeezed out when the intensity of pressure upon it was large. The manufacturers who employed it did so because they claimed that it was impossible to produce a close bearing in any other manner. Fortunately, the detail is now a thing of the past.

Truss swing spans are almost invariably of the through type, primarily because the deck is usually kept as close to the high water elevation as it is safe to go, and secondarily because even when the fixed spans of the bridge are deck, it pays to make the swing span through so as to let small craft pass beneath without the necessity for opening the draw to let them go by. A good example of this is the author's railway bridge over the Maumee River near Toledo, Ohio. It is so high above the ordinary stage of water that most of the passing craft go beneath it, thus saving the constant breaking of the railroad track which would have been necessitated had the swing span been made deck.

In respect to the power required to operate rim-bearing draw-spans, the author for many years has used an average of the Boller formulæ, viz.,

$$\text{H. P.} = \frac{0.0125\, Wv}{550}$$

where W = total load on rollers in pounds, and v = velocity on pitch circle of rack in feet per second; but in the specifications of Chapter LXXVIII (Clause 87) there is given a more detailed method for making the computation.

The author obtained a fine check on the correctness of the Boller formula when testing the draw-span of his Jefferson City highway bridge. This span of 440 feet weighs 660,000 pounds, and was opened by four men in four minutes and fifty seconds. The power applied by the men was measured by dynamometers, and from the length of their path and from their pull the horse-power was computed. It proved to be just a little less than unity; so near, in fact, that it was called unity. The velocity v was, on the average, 0.066 feet per second. Substituting in the formula gives

$$\text{H.P.} = 0.0125 \times 660{,}000 \times 0.066 \div 550 = 0.99.$$

It is possible that, if the experiments were to be made again, a greater divergence from the formula would be found, for the reason that the bridge is liable to work more easily after it has been operated a while.

Concerning the methods of computing live load stresses in swing-spans, the author, in 1897, wrote thus in *De Pontibus:*

> "Candidly, the author has very little faith in even the approximate correctness of the ordinary methods of computing live-load stresses in draw-spans; nor has he much more in the superrefined methods involving the principle of least work, or stretching of the different truss members, or the principle of the Three Moments with varying moments of inertia. In his opinion, there is but one satisfactory method of ascertaining the reactions for both balanced and unbalanced loads, viz., by making large models of a number of spans of various lengths, and weighing therewith the reactions for all kinds of loading. From a series of experiments of this kind there could be prepared a diagram or diagrams, similar to that shown on Plate IX, which would give approximately correct reactions for all spans and all loadings. Such an investigation would require considerable time and money; but if some professor of civil engineering would undertake to make the experiments, he could undoubtedly get the models built free of charge by dividing up the work among several of the leading bridge manufacturing companies. The results of such experiments would be of great value to both the engineering profession and the railroads of America."

Later the author persuaded his friend, Prof. Malverd A. Howe, the well-known engineering writer, to make the suggested series of experiments on end reactions from balanced live loads upon a rather-large-scale wooden model of a swing span having four points of support. Prof. Howe reported that all his experiments gave a phenomenally close agreement with the reactions as computed by formula and as indicated on the diagram just mentioned, which gives the proportions of end reactions for balanced live loads when the swing has four points of support. It is reproduced in Fig. 29*a*. That diagram was prepared in the late eighties by the author, in opposition to the advice of his then-associated engineers, who claimed that it would not be accurate enough for all cases, as it was based upon average relations of span and mid-panel lengths; but after they had tried it for use in computing swing-span stresses, they reported that it gave so close an agreement to the results of the formula that they were perfectly satisfied. Fig. 29*b* gives the

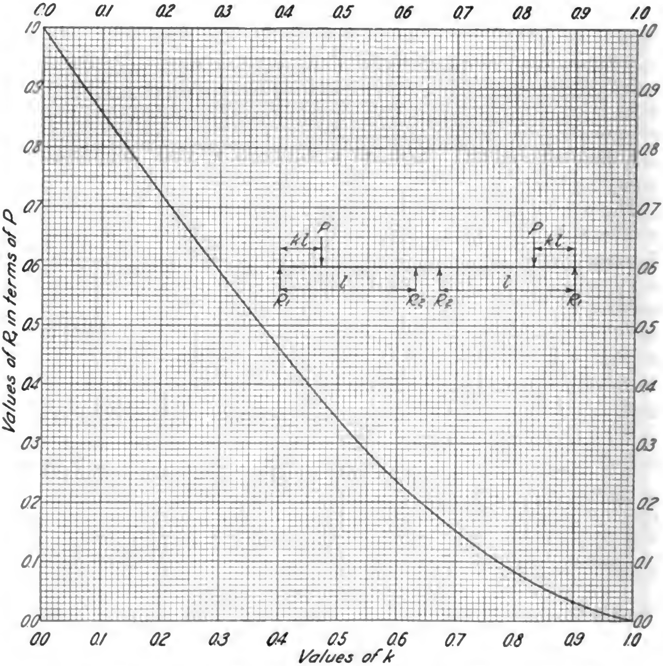


FIG. 29*a*. Reactions for Balanced Loads on Rim-bearing Draw-spans.

proportions of reaction for single loads in swing spans having only three points of support. The employment of these two diagrams will save the computer much labor in figuring live load stresses in rotating draws.

In determining the dead load stresses in swing spans, it is customary to assume that the draw is open; but the author also assumes, as previously mentioned in Chapter V, that there is an upward reaction from the lifting machinery at the ends, and finds the stresses therefrom; then, when any such stress tends to increase the section of any member, it is considered; but when it tends to decrease the section, it is ignored. This method may involve some errors on the side of safety, but they are of minor importance.

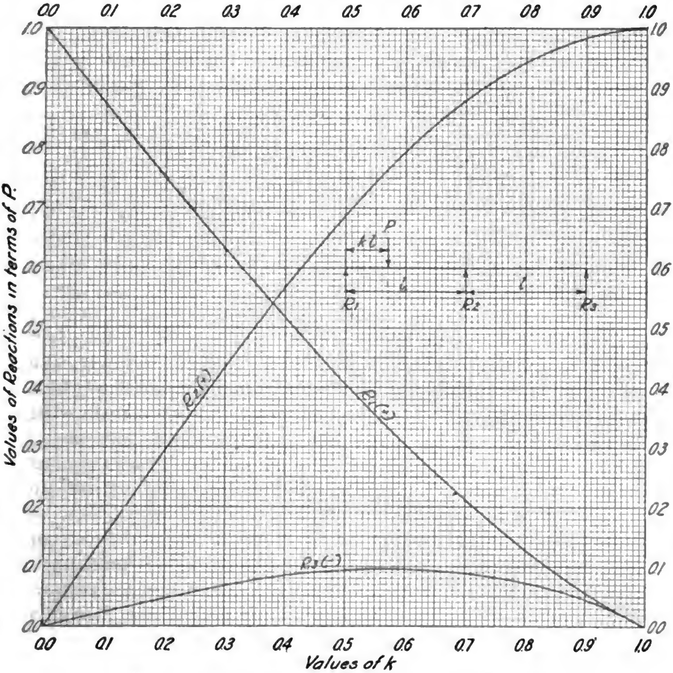


FIG. 29b. Reactions for Centre-bearing Draw-spans.

The designing of a drum for a turntable is a matter requiring much care. The load coming upon it should be distributed as much as possible, and all concentrated loads should be taken care of by a sufficient number of stiffeners of ample size and thoroughly attached. The girders over the drum should have not only ample strength and stiffening but also the proper comparative rigidities. The greater the number of points of support, the more evenly will the load be distributed to the drum and rollers; and the deeper the drum, the better the distribution. As an extra foot of depth of drum costs much less than one foot of height of pivot pier it stands to reason that it is better, whenever practicable, to make the drum much deeper than the calculations for strength and stiffness demand. The only good reason for not adopting in every case an excessive depth is that so doing might place the rollers below the level of high water, and thus render the structure liable to injury from drift, and the machinery to being blocked by an accumulation of mud under and between the wheels.

When the vertical distance between high water and the lowest part of the bottom chords is small, the longitudinal and cross girders can be placed with their bottom flanges flush with the lower surface of the said bottom chords, and the drum can be built inside of the box thus formed, so that its lower flange angles will be flush with the bottoms of the said girders. But if the vertical clearance be great enough to permit it, the box should rest on the drum at either four or, preferably, eight points.

Many designers rest the tower posts directly over the drum, thus making the diameter of the latter about forty per cent greater than the side of the square upon which are located the axes of the tower columns. Other designers let the sides of the square intersect the circle of the drum so as to divide the latter into eight equal parts, thus making the diameter of the drum about eight per cent greater than the side of the square. The author's practice for more than two decades has been to let the diameter of the drum equal the side of the square, obtaining eight points of support by inserting four small girders in the corners of the square, at angles of forty-five degrees with its sides. As the cost of a pier varies very nearly as the square of its diameter, it follows that this method of designing drums for rim-bearing swings effects a great saving in the cost of the pier as well as in that of the drum. Occasionally it will give a pier of very small diameter in comparison with the length of the draw-span. The remedy for this, provided the pier have the requisite stability against overturning, is not to increase the pier diameter but to anchor the draw-span to the pier in such a manner as not to interfere with the turning, but so as to offer an effective resistance to any tendency to lift the span off its support. In the case of the Jefferson City highway bridge, the length of the draw-span is four hundred and forty feet, while the diameter of the drum is twenty-two feet—the same as the perpendicular distance between central planes of trusses. Such a ratio of span length to drum diameter is too great for safety in case of a strong lifting wind acting on one arm only, for such an uplift would have to amount to only twelve and a half pounds per square foot of floor in order to throw the span off the pier.

It was, therefore, necessary to anchor the span to the pier by means of a long four-inch bolt passing through a wide heavy casting which is embedded in the concrete, and projecting at the upper end between two beams and through a saddle and a heavy washer-plate. The nut on the anchor-bolt is turned down so as nearly but not quite to touch the said washer-plate, thus causing no obstruction to turning the draw, but making the anchorage always ready to resist the slightest tendency to lift the span.

The limiting ratio of length of span to diameter of drum that can be employed without using a central anchorage cannot well be determined by rule, but must always be left to the judgment of the designer. It might suffice, perhaps, to specify that, whenever the uplift on one arm only necessary to upset the draw is less than fifteen pounds per square foot of floor in situations exposed to high wind-pressure, or less than ten pounds in other situations, an anchorage shall be adopted. In the case of three of the author's swing bridges on the Kansas City Southern Railway, the span length is two hundred and twenty-five feet, and the diameter of the drum is only seventeen feet; nevertheless no central anchorage was used. In these bridges the open floor reduces the uplift, and the situations are not such that the spans will be exposed to abnormally high wind-pressures.

Heavy draw-spans should be operated by two or more pinions; and when these are placed, as they should be, diametrically opposite each other, some kind of apparatus ought to be used to equalize the pressure on the pinions, otherwise both the latter and the rack are liable to have their teeth broken. The reason for this is that it is impossible to make the toothing of the rack so perfect in the distance of the semi-circumference that opposite pinions operated by a single shaft shall at all times act equally. When electrical machinery is used, the equalizing can be done by adopting independent motors; but with other machinery, some kind of mechanical equalizer should be employed. The author many years ago designed one for the first-built swing-span of the East Omaha Bridge, which worked to perfection. It was made by cutting the engine-shaft and attaching to each end a bevel-gear wheel. These bevel-gear wheels engage with two small pinions which are inserted between the spokes of a large spur-wheel that turns loosely on the engine-shaft. If we assume the pressures on the main rack-pinions on each side of the drum to be constantly equal to each other, the two halves of the engine-shaft will always have the same angular velocity; but in case the pressure on the teeth of the two rack-pinions on one side of the drum should fall below that on those of the two rack-pinions on the other side, the spur-wheel will move slightly on the shaft until the rack-pinions receive equal pressure again. By this apparatus equal pressure on the teeth of rack and pinions is at all times insured. The author was convinced of the necessity for such a device by watching it when the span was being turned; for several times during each quarter rotation the little pinions on the spur-wheel would make a sudden movement of such magnitude as to indicate a considerable variation in the spacing of the rack-teeth.

In designing draw-spans with high towers, especially long, double-track ones, there is an important matter that is sometimes overlooked, viz., the tendency of the end of the unloaded arm to rise when a moving load is on the other arm. For single-track bridges the only harm that this would do would be to pound the end bearings; but for a double-track bridge it would certainly some time cause a serious disaster by the derailment of an oncoming train when the other track on the other arm is covered by another train. Before designing the 520-ft. draw-span for the East Omaha Bridge (see Fig. 52*k*), the author looked up this matter as well as he could, having heard of trouble being experienced from rising ends on a double-track draw-span but little shorter than the one then contemplated. The results of the investigation were rather contradictory, consequently the design was made with three features that were conducive to resisting the raising of the ends, viz., extra-deep trusses at both inner and outer hips; stiff, continuously riveted top chords between these points; and an end-lifting apparatus capable of raising the ends one and a half inches. This was the best at that time which the author could do to avoid the difficulty; but at the same time he figured upon using later a holding-down apparatus in case the necessity therefor should ever arise. This span has at present only a single track at the middle, and the highway cantilevered floors are not yet put on. Observation has proved that, with one arm loaded by a train and the other arm empty, there was no rising of the ends when the latter were properly supported. Some years after the completion of the bridge as first built, an inspection showed that the timber cribs, which were then used as a temporary support for the swing span, had so shrunk vertically, on account of the seasoning of the wood, that the end rollers barely touched their bearings, necessitating some shimming thereunder. This condition of the ends afforded an excellent opportunity to note the rise with one arm only loaded by an engine and enough cars to cover the said arm. The amount observed was three-eighths of an inch. From this it may be concluded that with masonry piers and the completed superstructure, and with a hoist of one and a half inches by the lifting gear, there is no chance for the ends to rise from their bearings; for, to cause such a rise, it would take a live load just four times as large as the test load, which is more than could be placed on the double-track railway, wagonways, and footwalks. Had the bridge been built with shallow trusses and with eye-bars in a portion of the top chords between outer and inner hips, as was the similar bridge which was reported as giving trouble from rising ends, it is probable that similar difficulty would have been found in this structure.

Some engineers may think that, because each span of a draw is figured as an independent span for unbalanced live loads, on the assumption that the longitudinal tower rods are so small as to carry no vertical shear past the drum, there should be no tendency for the end of one arm to rise when the other arm is loaded; but such is not the case, as the tendency would exist if there were no longitudinal tower rods at all. The rising, for instance, of the right-hand end, induced by a live load on the left-hand arm, is evidently due to the fact that the inner hip of the left-hand arm moves to the left and downward a small amount. This movement causes the inner hip of the right-hand arm to move to the left and upward a similar amount, and, as a result, the end of the right-hand arm tends to lift.

In erecting draw spans, some method of adjustment must be provided so as to bring the ends to the correct elevation. This is accomplished by placing a group of thin plates under each bearing on the rest-piers. Two decades ago the metal manufacturers deemed it to be absolutely necessary in spans of more than 200 or 250 feet to provide also an adjustment for each bottom chord of each arm near the drum by inserting vertical transverse plates at the splice of the chord to the longitudinal girder over the drum. The sole reason for this detail was the crude shopwork of those days; but some twelve years or more ago when designing the second swing span of the East Omaha Bridge (then the longest draw span in the world, and exceeded today by only one span of a lighter structure that is one foot longer), the author, deeming that the shop work of the American Bridge Company had improved sufficiently to warrant the change, omitted the chord adjusting plates and relied entirely upon those under the bearings on the rest-piers. This required very careful calculations for deflection, because any material error might have put a break in the grade over the rest pier too great to work out by dapping the track ties. Fortunately, the experiment was a success, and ever since that time the author has followed in his practice the precedent thus established.

In all swing spans there must be some kind of arrangement for lifting the ends when closed. Numerous mechanical contrivances have been employed for this purpose, including rollers, wedges, screws, eccentrics, cams, hydraulic rams, and toggle joints. The requirements for a satisfactory lifting apparatus are as follows:

First: It should provide sufficient power to raise the ends to the required height within a reasonable time.

Second: The energy lost through friction should be a minimum.

Third: The resistance to the mechanical effort should be fairly uniform.

Fourth: The bearing afforded finally after the ends are raised should be solid and substantial, similar to the pedestals in a fixed span.

Fifth: When the span is closed it should be free to move longitudinally under changes of temperature.

The most common details for lifting the ends are the transverse roller, the longitudinal roller, and the wedge. The first mentioned was the one in general use until about twenty-five years ago, then the third gradually replaced it. The second mentioned device is not at all common, but has been employed for a number of years. Its advantage over the first type is that the actuating toggles are more conveniently placed, lying close to the bottom chords instead of beneath the end floor-beams. The wedge requires more power to operate than the roller but affords a somewhat better bearing. Bevels for wedges vary from one in ten to one in five, Schneider preferring the steeper pitch. The mechanism for moving the wedges should be designed so as to make the resistance to motion nearly uniform during all stages of the lifting, and so as to lock them in order to prevent their sliding backward. In small centre-bearing swings, especially in highway structures, it will suffice to have the lifting apparatus at one end only, thus producing a slight tipping of the span; but it is evident that such an arrangement would not suffice for a long span, because it would produce an unequal distribution of load on the rollers.

In the old forms of end lifts the nut traveling on the horizontal screw was of steel without bushing, and at times of heavy duty, especially when the weather was warm and the span deflected abnormally in consequence, this nut sometimes became welded to the screw; but bushing with phosphor-bronze has been found to stop this trouble entirely. In all swing-spans exceeding two hundred and fifty feet (or better still, two hundred feet) in total length there should be a nest of longitudinal rollers over each bearing on the rest piers so as to permit of the unimpeded expansion and contraction of the span. The roller nests, preferably, should be attached to the moving span instead of lying permanently on the piers.

In centre-bearing swings there are two methods of carrying the weight to the pivot, viz., by suspension and by superposition, the former being preferable. Its advantages are that it brings the point of support nearer to the centre of gravity of the bridge, that the disks can easily be removed, examined, or replaced without interfering with traffic, and that it provides an easy method of adjusting the height of the span. It is best to use phosphor-bronze disks between two hardened-steel disks; for the surfaces of the latter in contact with the phosphor-bronze cannot wear out, consequently the wearing is confined to the alloy disk, making it the only part outside of the operating machinery which will ever require replacement.

All railroad swing-spans must be provided with some kind of device for lifting the rails, in order to permit them to swing clear of the approaches when the span is rotated. That designed by the late Geo. S. Morison, Past President of the American Society of Civil Engineers, has been used very generally.

As indicated in the preceding chapter, just beyond each end of every swing-span (or of any other movable span) for highway traffic there should be provided a substantial and quickly operated gate or portcullis for the prevention of accidents due to animals or vehicles running off the open end of the approach. Failure to supply such a device has already been the cause of the loss of many lives and much valuable property.

The tops of all pivot piers should be so designed as to drain thoroughly by pitching the upper surface from the centre toward the periphery, and by providing an adequate number of weeping pipes that pass below the lower-track segments.

In designing all parts of the turntable, the operating machinery, and the girders over the drum, great care is necessary to ensure that every piece and every connection are made sufficiently strong and stiff; for there are involved certain bending moments, torsions, and secondary stresses that used often to be overlooked, the result being loosened connections, broken rivets or bolts, and machinery out of order. The truly scientific designer nowadays will give due consideration to all these unusual conditions and will meet them by using ample sections for all parts and a liberal supply of rivets in the connections. The attachment of gear-brackets to the drum used to be the detail that gave most trouble, because of the great bending moment induced by the turning of the down-shaft when the span was operated against a strong, unbalanced wind pressure.

All man-power machinery should be made very strong, because if anything prevents the apparatus from operating properly, the men are likely to crowd upon the levers wherever they can find room and surge thereon to their utmost capacity. Once when operating by hand the first-built swing-span of the East Omaha Bridge, using two sets of six or seven men on each of the two four-armed levers, it failed to move. Immediately upon finding the unexpected resistance, they all stepped back a few feet and threw themselves with full force upon the levers. the result being the same as before. The author stopped this instantly, and upon investigating found that the two sets of men were working against each other. By starting one set in the opposite direction the span was readily put in motion. This example is given to show how ignorant workmen will abuse machinery, and the consequent necessity for making all man-power apparatus extra strong, notwithstanding any opposition that may be offered thereto by bridge manufacturers or other interested parties. If the specifications given in Chapter LXXVIII be strictly adhered to, and if due consideration be given to all the existing conditions when designing operating machinery, ample strength, rigidity, and endurance will be attained without any great unnecessary expenditure of metal or shop work.

As a drawbridge is a piece of machinery, it will require a certain amount of care, for otherwise it will get out of order and give trouble just at the wrong time. It should be opened at least once a month, and all parts which move on other parts, especially the wheels and tracks, should be kept clean and well lubricated. The lower rolling surface for the wheels should be kept free from all obstructions, and the wheels should be maintained in proper adjustment. The operating machinery also should receive due care and attention. In respect to those details of design of swing spans which affect materially the question of maintenance, Mr. Cartlidge has expressed his opinion in his before-mentioned paper, and as the author concurs in it without exception, it is herewith reproduced as follows:

> "It may be laid down as a general rule that there should be absolutely no adjustable members in the trusses. All parts subject to complete reversal of stress should be stiff members and have, as far as possible, riveted connections. No pin-connections should be employed save for eye-bar members. This is particularly true of the connection between the end of the lower chord and the foot of the end post. The constant reversal of stress at this point, due to lifting and lowering the ends of the draw, very soon develops serious wear on the pins and pin-holes. With a properly designed riveted connection, no play being possible, there will be no difficulty.
>
> "In draw-span design, perhaps to a greater degree than in any other, simplicity and rigidity are the prime requisites to economical operation and maintenance."

In making preliminary estimates for the cost of bridges on a railroad the question sometimes arises as to how the total weight of metal in a swing span, including that of the operating machinery, compares with that in a simple span of the same total length. This question cannot be answered with accuracy, mainly on account of the personal equation of the designer; but, in general, it may be stated that for spans of one hundred feet the weights are about equal, and for spans of five hundred feet the swing with its machinery requires seventy-five or eighty per cent of the amount of metal in the fixed span, as can be seen by referring to Fig. 55*ee*.

But if the question be one of comparative costs of the superstructures of swings and fixed spans, that is quite a different matter; because the machinery metal of the former in place is about two and a half times as expensive per pound as the ordinary structural steel, making the average pound price for the erected metal of a draw from one and a half cents to two cents higher than that of the corresponding fixed span. Again, the preceding figures do not allow for the cost of electric motors or gasoline engines; hence it is evident that the total cost of a swing span is always greater than that of a simple span of the same length. If the operation is to be done by man-power, the ratios of total costs will vary from 1.4 for spans of 200 feet to 1.13 for spans of 500 feet. If the cost of mechanical power be included, these figures would be about 1.5 and 1.2.

The economics of swings as compared with other kinds of movable spans are treated in Chapter LIII, and the specifications for designing them are given in Chapter LXXVIII.

In Chapter LV there are given directions for finding the weights of metal per lineal foot of span for the various portions of swing bridges and for the spans as a whole.

In Chapter LXXVIII there is given, in the portion of the specifications relating to draw-bridges, much information concerning styles of bridges for various span lengths, heights of towers, depths of trusses, panel lengths, loadings of all kinds, combinations of stresses, details of design for various styles of swings and their turn-tables, operating machinery, power determination, machinery houses, etc.; and the reader who has a swing span to design is advised to read the same with care before starting his computations.

CHAPTER XXX

BASCULE BRIDGES

The modern bascule span has for its prototype the drawbridge of the mediæval castle. In ancient times it served a double purpose—bridging the moat when lowered and barricading the doorway when raised. It was hinged at one end and raised by hand power; and, consequently, only short light spans could be utilized. Although these early bascules were counterweighted to some extent, the simple arrangement of weights attached to chains running over pulleys and connected to the free end of the span did not provide a balanced system and, therefore, it was hard to start the bridge and hard to check its motion when nearly raised. In this regard these early types did not measure up to the significance of their name—"bascule" coming from the French and meaning a balance.

Owing to the crude arrangements of counterweights and the lack of ample and convenient power for operating, the bascule remained in its primitive state until comparatively recent years. Most of the early types rotated about a fixed axis. Two exceptions were the 40-foot track girder bridge built at Havre, France, before 1824, and another, rotating on a wheel, built at Bregère. These were the forerunners of the modern rolling lift bascule. An early span of the trunnion type which gave practical service was the railroad bridge on the line of the North Eastern Railway at Selby, England. This bridge was built in 1839, and consisted of two fixed spans and two moving leaves which gave a 45-foot clear channel when opened. When closed the bascules formed an arch. For operating them a rack wheel and hand power gearing were employed. Another trunnion bascule was the Knippel bridge built at Copenhagen in 1867. Hydraulic power was used for operating this span, and cast iron pockets were provided for the counterweights, which were attached directly to the short arm of the rotating span and thus maintained a uniform balance. In 1878 the Fijeenord trunnion bascule was built at Rotterdam, Holland. It has a clear span of 75 feet. Each leaf is in two sections with four trusses to a section. The two outside trusses act as arches when the bridge is closed, while the two inner trusses carry counterweights on their short arms. The gearing can be operated by gas, hydraulic power, or man power. Another trunnion bascule is the highway bridge built at Koenigsberg in 1880. This bascule acts as a cantilever when closed, anchors being provided at the piers to take care of the uplift on the tail ends of the leaves.

The first important bascule bridge built in the United States is the

Michigan Avenue Bridge at Buffalo. This is of the trunnion type with cables attached near the free end and running diagonally to pulleys at the top of the tower, over which they pass to large, cast-iron wheel-counterweights. The latter roll on a specially curved track so constructed that the component tension in the cables decreases as the lever arm of the centre of gravity of the leaf diminishes. Several bridges of this character were built; but, other types proving more efficient, their construction was discontinued.

The modern era of bascule building may properly be said to have commenced with the construction of the Tower Bridge of London in 1894. At the same time the Scherzer rolling lift bascule was completed for the Metropolitan Elevated Railroad over the Chicago River near Van Buren Street. Since that time the bascule bridge has developed rapidly, and many different types, or rather sub-types, have been brought out. Whereever heavy bridge traffic has to be frequently interrupted by boat service the swing bridge is no longer adequate, and the bascule bridge becomes one of the alternatives for the engineer to consider. The advantages of the bascule over the swing span are:

1. Wide centre channel free from piers and pier protection.
2. Increased space for dockage.
3. Rapidity of opening to permit passage of vessels and subsequent closing again for bridge traffic.

For a general discussion of the comparative advantages and disadvantages of the bascule with other forms of movable bridges the reader is referred to Chapter XXVIII.

Modern bascules are comprised in three classes, viz., the trunnion type, the rolling lift type, and the roller bearing type. Any of these bridges may have either a single leaf or two leaves meeting at the centre of the span. For railroad traffic the single leaf is preferable, for it can be made to act as a simple span when closed; and greater rigidity is thereby secured.

In the trunnion type the centre of rotation remains fixed or nearly so, and is at or close to the centre of gravity of the rotating part. This is a highly desirable feature where yielding foundations are unavoidable. In the rolling lift type the centre of rotation continually changes and the centre of gravity of the rotating part moves in a horizontal line, thereby shifting the point of application of the load on the pier, which is a faulty feature, unless the pier be founded on rock. The rolling lift in opening recedes from the channel, thereby leaving a greater clear waterway for the same span length than does the trunnion type. However, it also encroaches on the land side, which is objectionable in congested quarters. In the roller bearing type the centre of rotation remains fixed and coincides with the centre of gravity of the moving mass. The trunnion is eliminated and the load is carried by a segmental circular bearing on rollers arranged in a circular track. In this way the load can be distributed over greater area, thereby reducing the unit bearing stress; and at the same time the frictional resistance to rotation is decreased.

Much ingenuity has been exercised in devising various mechanisms and operating machinery in the attempt to overcome the several unsatisfactory features of the original bascules. This has led to different subtypes or varieties. To the rolling lift class belong the Scherzer and the Rall varieties. To the trunnion class belong the Strauss, Brown, Page, Chicago City, and Waddell & Harrington varieties; and to the roller bearing class belong the Montgomery Waddell and the Cowing varieties.

In the Scherzer bascule (see Fig. 30*a*), the leaf, L, rotates on the quadrant Q, which rolls along the horizontal track girders, T. The centre of gravity, G, of the leaf is at the centre of this quadrant and, therefore, moves in a horizontal line as the bridge opens. A counterweight, W, is attached to the short arm projecting shoreward, so that the leaf is maintained in balance at all positions; and, consequently, the operating machinery has only to overcome inertia and the friction of the moving parts. A pit, P, is provided in the main pier so that the counterweight can sink into it as the leaf opens and rolls backward. This pier is of large size, as it carries the track girders; and it requires a good, solid foundation, since the shifting of the point of application of the load disturbs the base pressures. Two other smaller piers are required for a single leaf structure—a rest pier at the front end and a shore pier or abutment at the rear end to carry the approach span. In the case of a double leaf bascule a second main pier will be required and also an abutment. A locking device at the centre of the span connecting the two leaves when the bridge is closed renders unnecessary a rest pier. The span is operated by a pinion working in a rack pivoted to the upper part of the quadrant. Fig. 30*b* shows one of the Scherzer rolling lift bridges.

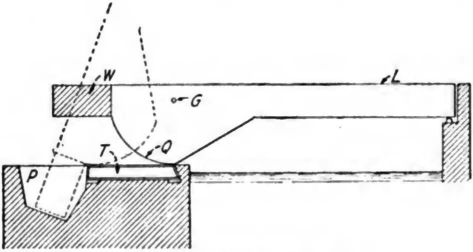


FIG. 30*a*. Scherzer Bascule.

The Rall type, shown in skeleton form in Fig. 30*c*, rotates about the centre of gravity, G, of the leaf where a pivot or trunnion is provided, which rests in a roller, R, carried by a horizontal track girder, T. When the leaf is closed the main girder or truss bears on the pin A, which is fixed to the pier; and the roller, R, is slightly raised off the track girder, so that the load on the bridge is carried directly by pin A to the pier. The swing strut, S, is connected at one end to the movable girder by pin B, and at the other end to pin A. When the leaf rises, it first revolves around pin A, until the roller is in full bearing with the track girder; then as the operation is continued, the roller moves horizontally on the track girder, while pin B of the main girder describes an arc with A as the centre. The leaf is operated by the main pinion, P, engaging a rack fixed to the strut, E, which is pivoted to the girder at C. When the leaf is closed the pivoted roller, R, is free and can be removed and replaced without difficulty. The centre of rotation is so far above the pier that no pit is required to receive the tail or the counterweight, W. The horizontal motion of the pivot is sufficient to allow the tail to clear the masonry when

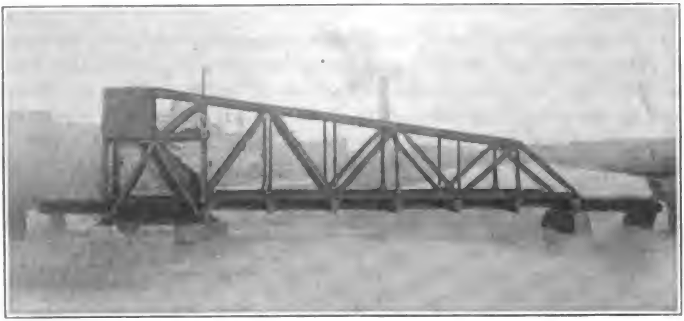


FIG. 30*b*. Scherzer Bascule Bridge.

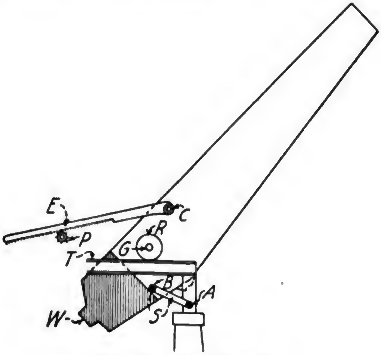


FIG. 30*c*. Rall Bascule.

the span is raised. This retreating motion of the leaf permits of using the minimum span length to obtain a given clear waterway. However, the shifting of the centre of gravity disturbs the foundation pressures. Fig. 30*d* shows the Rall bascule erected at Peoria, Ill. The Rall bascule patents are now controlled by the Strobel Steel Construction Company of Chicago.

The distinctive feature of the Strauss trunnion bascule is the pivoting of the counterweight at the end of the short arm. This enables the said counterweight to move parallel to itself at all times; and it can, therefore, be made in such shape that no pit is required to receive it when the leaf is in an upright position. In one variety of this type the counterweight is placed beneath the approach floor. In the other variety it is located in a frame above the floor (see the skeleton diagram in Fig. 30*e*). The frame, F, carrying the counterweight, W, is attached by means of a strut to the short arm of the bascule at the pivot, A, and at the top by the pivot, B, to a link, K, which is pin-connected to the tower at C. This system of connections provides for a parallel movement of the counterweight at all times, and thus does not alter the ratio of lever arms nor displace the centre of gravity of the system, which is at the main trunnion, G, of the bascule. The leaf is operated by the pinion, P, engaging the rack, R, on the short arm. Fig. 30*f* illustrates the Strauss bascule at Polk Street, Chicago. Since the construction of this bridge, the Strauss Bascule Bridge Company has developed a modification known as the "heel trunnion" bascule, which is shown in skeleton form in Fig. 30*g*. This modified type has a fixed pivot point, E, at the end pin of the bottom chord of the truss. The counterweight trunnion, T, is also a

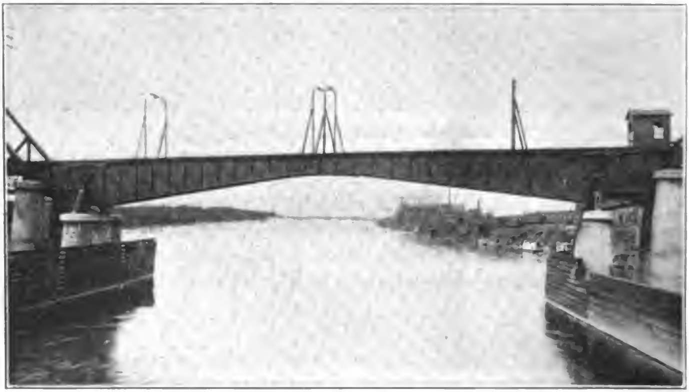


FIG. 30*d*. Rall Bascule Bridge over the Illinois River at Peoria, Ill.

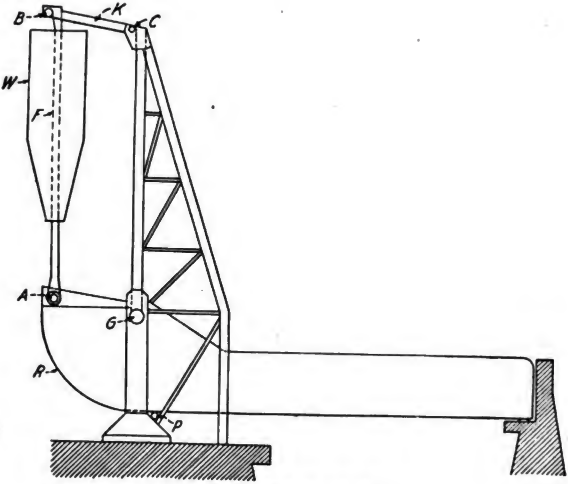


FIG. 30e. Strauss Bascule.

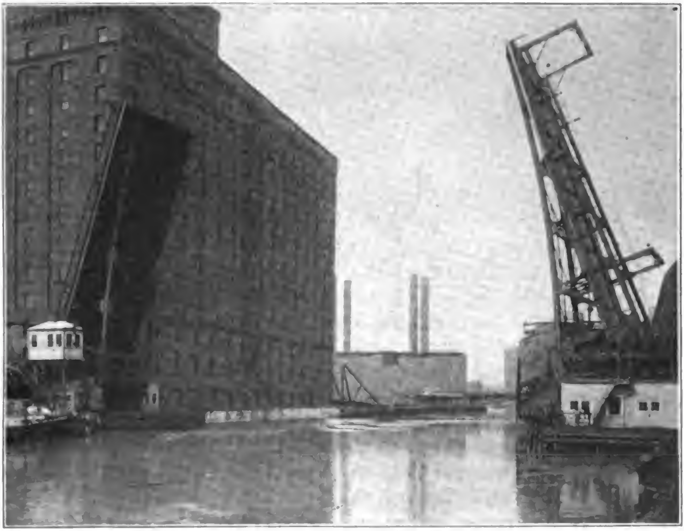


FIG. 30f. Strauss Bascule Bridge at Polk Street, Chicago, Ill.

fixed pivotal point, and is located at the top of a stationary tower supported by the main pier and an auxiliary pier. The counterweight, W, is carried by one end of a trussed frame rocking on the trunnion, T. The other end of this frame is connected by a pivot, F, to a link, K, which in turn attaches to the hip of the main truss by the pin, H. This provides a parallelogram of linkages, with the side formed by the triangular tower before mentioned as a fixed link. Near the centre of the latter the operating strut is pivoted by the pin, C. A pull on the strut, S, causes the parallelogram to close up, thereby raising the leaf. A detailed description of the heel trunnion type will be found in *Engineering News*, Vol. 67, page 830.

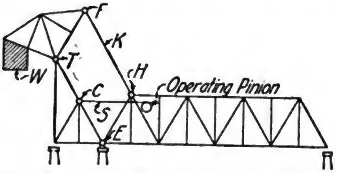


FIG. 30*g*. Strauss Heel-trunnion Bascule.

The Brown type of trunnion bascule differs from the others chiefly in its method of operation and in the application of its counterweights. (See Fig. 30*h*.) The usual truss form rotates about a pivot, E, in the end post. Two connecting links, K and J, control the movement of this point. The link, K, is hinged at H, a fixed point on the approach girder, G, while the other link, J, is really a continuation of the end post and is connected by a pin, P, to the cross-head at the end of the piston rod of a hydraulic cylinder, C. This cross-head slides along a horizontal guide, T, and transmits through a strut, L, which is really a continuation of the lower chord, its motion to the span. As this cross-head moves forward, the leaf is forced to rotate about the pivot, E, as that point is fixed horizontally by

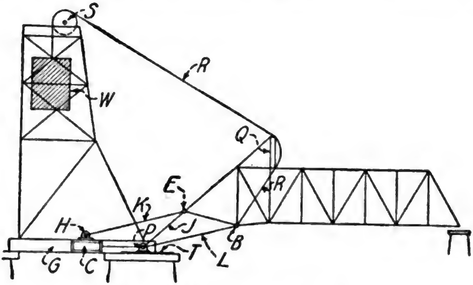


FIG. 30*h*. Brown Bascule.

the link, K, the said link being the only member that can provide the reaction to the force on the pin, P. However, this system of linkages provides for a slight, nearly vertical motion of E as the span rises, the link, K, turning about the hinge, H. The fixed length of the link, J, and its attachment to the cross-head held to the guide, T, limit the movement of E to a small arc only. The counterweight, W, is of the overhead type, and moves vertically in a tower built over the approach span. It is attached to a cable, R, which runs over a sheave, S, at the top of the tower and then on an inclination to a specially curved and grooved track, Q, fixed in an upright position to the top chord, around which track the cable bends to a reverse inclination and then fastens to a pin, B, at the panel point in the lower chord next to the end. The curvature of the track, Q, is such that, as the span rotates from the horizontal position through an angle of about 81 degrees, the horizontal reaction on the cross-head remains very small (thus ensuring that the mechanism will have to overcome merely the friction of moving parts and the wind pressure), while the vertical reaction thereon gradually increases from zero to an uplift of nearly one-third of the weight of the span. After a rotation of 81 degrees has occurred, the centre of gravity of the span is almost directly over the point P, and the line of action of the cables

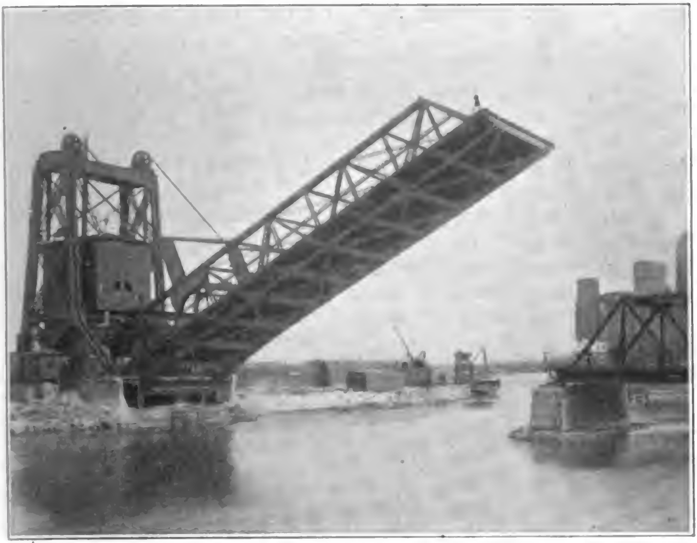


FIG. 30*i*. Brown Bascule Bridge at Buffalo, N. Y.

passes through the same point. If the movement progresses still further, the cables leave the guide, Q, and come into contact with other guides (not shown in the sketch) located near the top chord; and the bending of the cables around these guides sets up a horizontal force which prevents the span from tipping over toward the tower. During this last stage, the horizontal reaction on the cross-head increases very rapidly, while the vertical reaction thereon remains nearly constant. Fig. 30*i* illustrates the Brown bascule at Buffalo, N. Y.

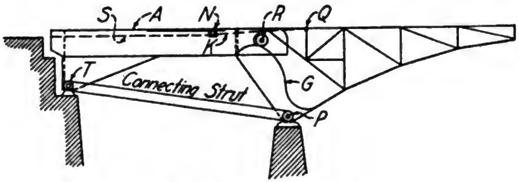


FIG. 30*j*. Page Bascule.

The Page bascule has the unique feature of a tilting approach span for highway bridges. This approach span is utilized as a counterweight. In through railroad bridges the approach span is fixed and a tilting counterweight is placed overhead. As the principle of operation is the same in each case, one description will suffice for both kinds. See Fig. 30*j*. The approach span, A, pivots on trunnions, T, at the shore end; while the free end is carried by rollers, R, resting on specially curved track girders, G, that are fixed to the main trusses of the bascule. As the leaf rises the track girders rotate with it about the pin, P, and cause the free end of the approach span to drop also. This approach span is loaded so that it balances the weight of the leaf in all positions. To produce this condition of constant equilibrium, the contour of the track girders, G, is curved in such a way that the point of application of the counterweight load gives a decreasing lever arm as the leaf rises and its centre of gravity approaches the vertical line passing through the centre of rotation. The operating mechanism consists of long screws, S, provided with nuts, N,

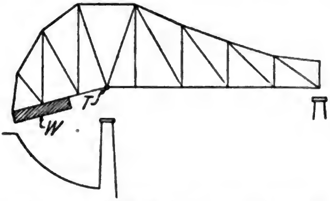


FIG. 30*k*. Chicago City Type Bascule.

moving in guides on the girders. The motion of the nuts is transmitted by the operating struts, K, to the truss through the pin connection, Q. Owing to the inherent inefficiency of the screw mechanism, more power is required to operate this type than is needed for any of the other bascules. The effectiveness of the counterweight is reduced by the support given the counterweight girders at the pivot, T. No pits are required to receive the counterweights.

The Chicago City Type of bascule was developed by the Engineering Department of Chicago. See Fig. 30*k*. The trusses are supported on trunnions, T, in line with the lower chord, placed a short distance back from the centre of gravity of the span. Counterweights, W, are rigidly attached to the end of the shore arm, and a pit is provided in the pier for their reception when the bridge is opened. The leaf is operated by a pinion and segmental rack attached at the end of the short arm. Elastic bumpers are provided to absorb the shock in opening and closing the span. A worm gear brake is also supplied to check any downward motion of the leaf, should occasion require. For a double leaf bridge centre locks are employed, but no rear locks are needed, as the centre of gravity is ahead of the pivot.

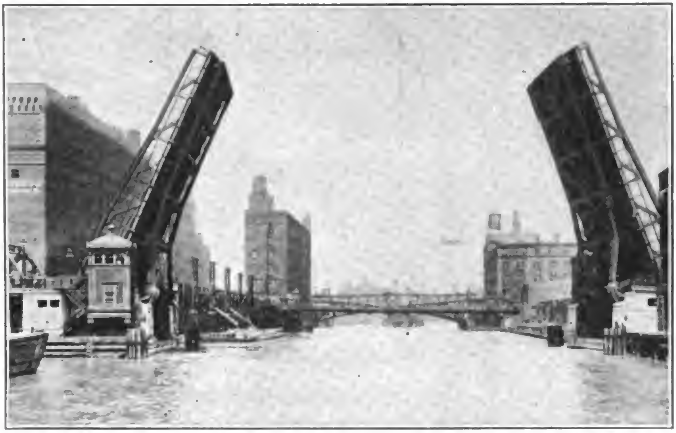
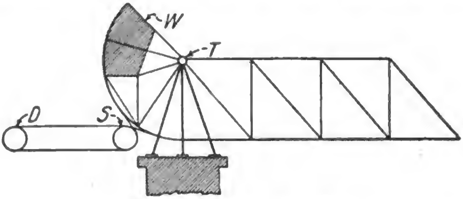


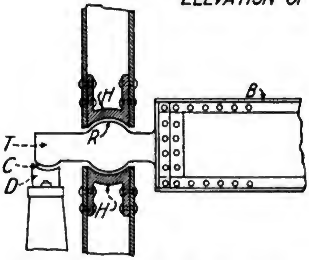
Fig. 30*l*. Chicago City Type Bascule Bridge.

Fig. 30*l* shows a Chicago City Type bascule bridge opened for river traffic. Just beyond can be seen a Scherzer Rolling Lift Bridge.

The Waddell & Harrington bascule has a number of distinctive features. See Fig. 30*m*. The trunnions, T, which are in line with the top chords of the trusses, are made of special steel castings which are rigidly attached to a box-girder, B, spanning the distance between the trusses. The free end of each trunnion has a cylindrical bearing, C, with its axis parallel to the plane of the truss. This bearing fits into a cup, D, mounted on a standard or tower anchored to the pier. The object of this cylindrical bearing is to permit a slight rotation due to the deflection of the box-girder connecting the trunnions. Between this cylindrical bearing and the end of the box-girder is an enlargement of the trunnion, or a segmental ring, R, having a spherical surface. A hub casting, H, bored to fit this spherical surface, turns on the segmental ring and supports the truss. This gives a bearing of large area and permits of using a lower unit pressure. This spherical surface also provides for the slight bending of the trunnion in a plane perpendicular to the truss as the deflection of the box girder varies with the change in loads, thereby preventing any binding or any unequal distribution of loading on the two sides of the truss that would involve high secondary stresses. The span is operated by a system of cables connected by equalizer bars to each truss at the ends of the segment of the short arm of the bascule. These cables follow the curve of the segment and pass around a nearby idler sheave, S, under the floor and then to the winding drum, D, from which they return to the idler and then to the other attachment at the segment. Provision is made for reversing the rotation of the winding drum. As the span is



ELEVATION OF BRIDGE



ENLARGED DETAIL OF TRUNNION "T"

FIG. 30*m*. Waddell & Harrington Bascule.

balanced about the centre of rotation by a concrete counterweight, W, at the upper end of the segment, extending from truss to truss, only sufficient power to overcome the friction and inertia of the moving parts is needed to operate the span. Fig. 30*n* depicts a bascule of this type erected over False Creek at Westminster Ave., Vancouver, B. C.

The first roller bearing bascule was developed by Montgomery Waddell, Esq., C. E., to whom patents were issued in 1899. See Fig. 30*o*.

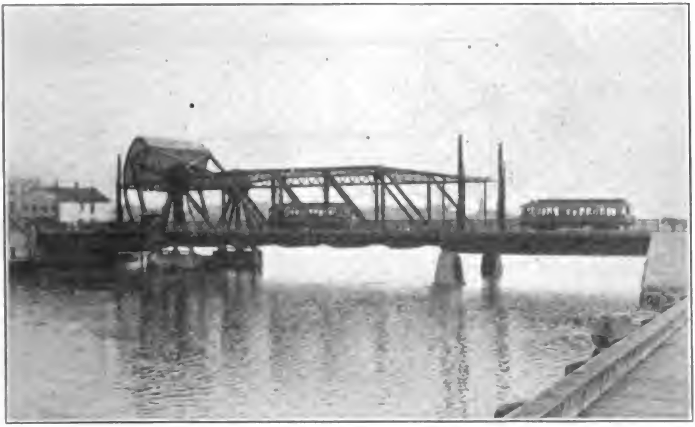
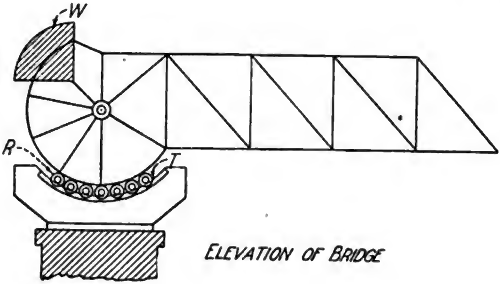
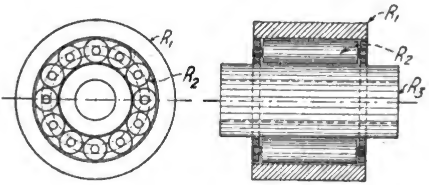


FIG. 30*n*. Westminster Avenue Bridge over False Creek at Vancouver, B. C.

There are two distinct designs for this type of bascule. In one the circular end of each truss of the moving span rests on a nest of solid rollers, R, that are effectively connected to each other by spacers and which are supported in a cylindrical, cup-shaped bearing. These rollers have trunnions which rest on the curved track, T, and which have a diameter one half of that of the rollers, consequently the translation of the rollers is only one-fourth as rapid as that of the cylindrical surface which bears on them. In the other design the last mentioned surface rests on two stationary compound rollers per truss, of the type shown in the lower portion of Fig. 30*o*. In both types, and more especially in the second, the frictional resistance to motion is reduced to a very small quantity. As shown in the drawing, the compound roller consists of a single large solid cylinder, R_3, surrounded by a nest of small, solid rollers, R_2, that are encased by a large, hollow cylinder, R_1. Such a combination approximates closely in efficiency to a ball-bearing. To operate the bascule, a pinion engages a rack on the outside of the segment in the planes of the trusses. An overhead counterweight, W, is provided at the upper end of the segment. No pit is required in the pier to receive either the tail end of the span or the counterweight. The centre of gravity corresponds to the centre of rotation so that only friction and inertia have to be overcome. Fig. 30*p* shows a general plan for a 750 foot, double-leaf, bascule bridge for a proposed crossing of the Mississippi River just below New Orleans, designed jointly by the author and his brother, Montgomery, for the noted railroad builder, the late Collis P. Huntington, Esq., and his consulting engineer, Dr. Elmer L. Corthell. The death of Mr. Huntington was the sole reason for the failure of this bridge project to materialize. In this case the rollers were to be stationary, and the counterweights were to be attached to long arms extending beyond the rolling segment and outside thereof. Fig. 30*q* shows a plan for a double-leaf bascule bridge over the Chicago Drainage Canal. For this bridge the moving rollers were selected. Attention is called to the relatively small amount of concrete needed for substructure.



ELEVATION OF BRIDGE



ENLARGED DETAIL OF A COMPOUND ROLLER

FIG. 30*o*. Montgomery Waddell's Roller-bearing Bascule.

The Cowing bascule, based on patents issued to John P. Cowing, Esq., in 1900, is very similar to the Montgomery Waddell type. The semicircular segment, forming the tail end of the lifting span, moves on a nest of solid rollers, which in turn move on a track girder curved to correspond with the said rolling segment. The counterweight is partly above the floor and partly below. The leaf is balanced in all positions,

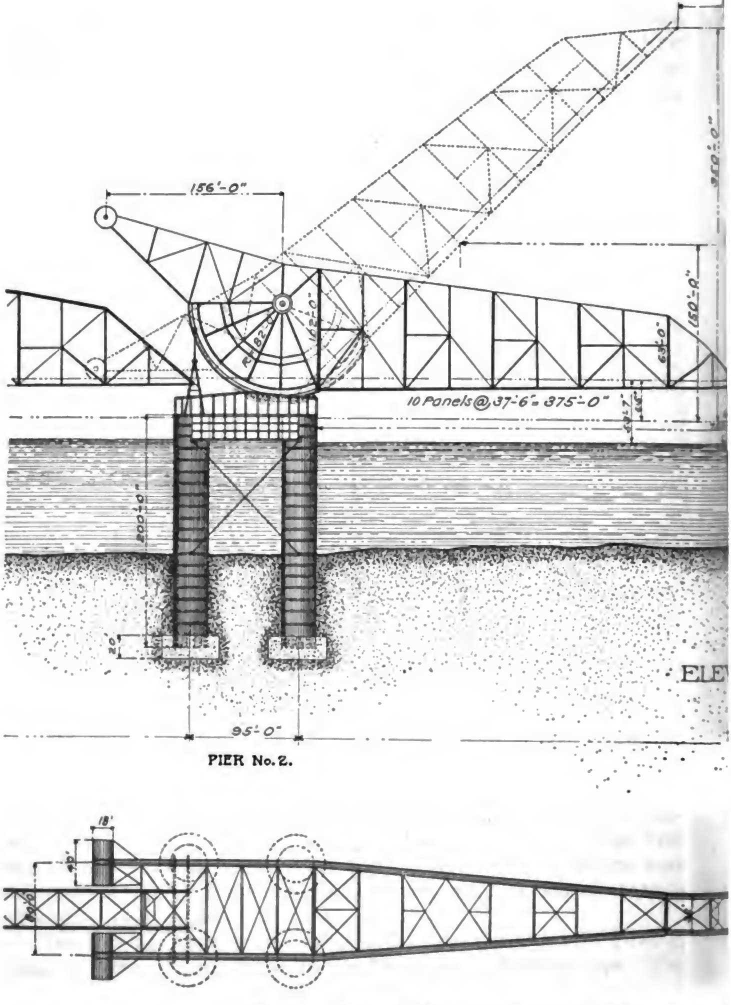
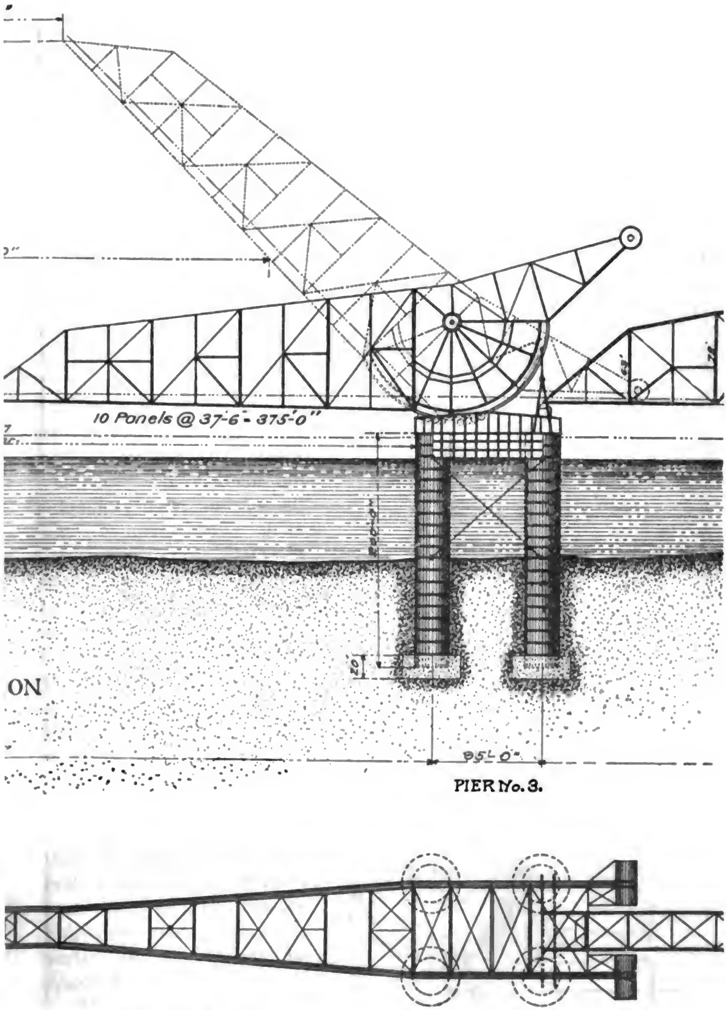


Fig. 30p. Proposed Roller-bearing-bascule Bridge over th



ppi River at New Orleans, La.

as the centre of rotation is at the centre of gravity of the mass. When the bridge is closed, the live load reaction comes on a bearing placed upon the pier in front of the curved track or cradle. It is claimed that the Cowing type is a direct infringement on the Montgomery Waddell patents.

The question is often asked: "Which is the best of the various types of bascule?" It is a difficult one to answer. Truth to tell, there is not today much difference in efficiency between any of them. Each has its advantages and its disadvantages. All of them are inherently ugly, and for all but comparatively short spans are uneconomic in comparison to the vertical lift; but they are scientific, and they represent, probably, the best and most profound thought that has ever been devoted to bridge engineering. They certainly are complicated structures, and as such they require good care and constant attention. They are more satisfactory than the swing span in several important particulars; and wherever they can be built more cheaply than the vertical lift, they should be adopted.

The retreating type, in which the axis of rotation has a motion of translation longitudinally with the struc-

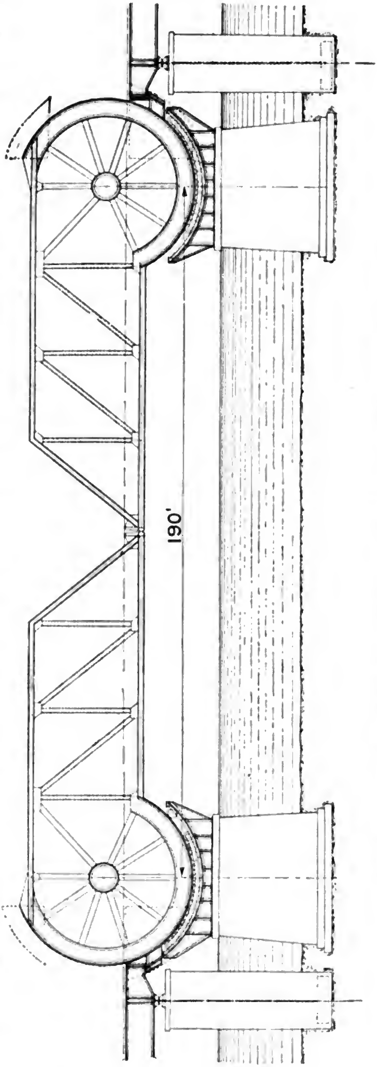


Fig. 309. Proposed Roller-bearing Bascule Bridge over the Chicago Drainage Canal.

ture, has the advantage of giving generally a greater clear opening for any total length of span than does the type with the fixed trunnion, and on that account it ought to be somewhat cheaper; but, on the other hand, it usually involves greater expense for the machinery and the structural details directly connected therewith. The principal disadvantage of the retreating type, as before stated, is that it is really suitable only when the pier foundations are either rock or some other very solid material; because the variation of the loading on a pile foundation, by constantly changing the location of the centre of pressure of the load, tends to rack the pier or abutment. In general, the author's choice would be for the type with the fixed position of axis of rotation, but he is not at all prejudiced either pro or con; hence, if a case were to occur in which the piers were designed to rest on a solid bearing and in which the retreating type showed by careful estimates a material saving in first cost, he would not hesitate to adopt it.

The Scherzer type of bascule apparently has been the most popular of all types up to the present time, notwithstanding the fact that many of the earlier bridges of this make wrenched themselves to pieces, the principal points of failure being the teeth of the rack, the peripheral segment connecting thereto, and the attachment of the said segment to the span; for the teeth broke and the rivets sheared constantly after a few years of service. It is claimed, though, that these defects have been remedied in the later designs of the Scherzer Bridge and that the structures of that type built during the last few vears are giving good satisfaction.

Not enough bridges of the Rall patent have been built to warrant one in passing judgment upon its merits and demerits; but, from all that can be learned, it appears to be a satisfactory type of structure.

A good many bridges of the Strauss type have been built, and from all that can be learned they are operating well; but they are specially deficient from the æsthetic point of view. However, that cuts very little figure, as no bascule bridge ever designed can be claimed to be a thing of beauty. If it will open quickly and keep in good order, that is about all that can be expected of it.

As yet there is only one of the Brown bascules in operation. It is giving good service and is of as scientific construction as any of the types. Some engineers may claim that the employment of wire rope in its design is a defect; but the author does not agree with that opinion, because that material is the most reliable of all the kinds of metal with which an engineer has to deal. For comparatively short span bridges (and those are the only ones for which the bascule is truly suitable) this type ought to give satisfactory service.

The Page bascule has not yet been thoroughly tested by age, nor have many of them been built. The screw mechanism employed in its operation is certainly not a feature of design that can be considered in its favor; for it must increase greatly the amount of power required.

The Chicago City type of bascule has given good satisfaction for a number of years, although it is said to be somewhat expensive in construction. The fact that it involves the use of a pit below water level may account for a large portion of the excess cost; and, moreover, that feature is not, to say the least, a desirable one, owing to the necessity for keeping the pit clean and free from water, However, the city authorities in Chicago seem to prefer it to all other types, possibly because it is not patented; and the existence of such a preference is certainly a strong point in its favor.

There has been but one bridge of the Waddell and Harrington type of bascule built, and that one does not often have to be operated—in fact, it is needed so seldom that the bridge has to be opened occasionally in order to keep all the moving parts properly lubricated. On that account it is impracticable to pass judgment upon its efficiency. This, however, may be stated—that, because of its two adjustment-provisions for axle deflection, it is the most perfect of all bascules yet built with overhead axle. The secondary stresses that are induced in any bridge of that type in which the axle deflections are not properly cared for would startle by their magnitude any computer who might take the trouble to analyze them thoroughly. The type is not specially economical. It was adopted by the City of Vancouver because of the fact that no royalty was asked for its use, the patents on it being controlled by the City's consulting engineers.

Of all the kinds of bascule with which the author has ever had anything to do, that one of his brother's numerous patented types which is described herein appeals to him the most, notwithstanding the fact that in years long gone by he made a number of unsuccessful attempts to introduce it in competition. His failures cannot be attributable to any inferiority in the plans or in the type of structure; for his estimates, as compared with those for the competing structures, always showed a decided economy in first cost. The true reason was that he was unwilling to resort to the means of introduction of the type that were indicated to him as necessary to ensure success. There is but little to choose from between the two styles of rollers, although Mr. Cowing evidently preferred the nest of solid ones, as that is the type which he adopted in taking out his patent.

More bascules of the trunnion type have been built than of the other types. The longest bascule bridge yet constructed is one of the Strauss trunnion variety at the Canadian Pacific Railway crossing of the Sault Ste. Marie ship canal. This bridge is of two leaves and has a length of 336 feet between trunnions. It is of the through truss type, and is provided at the ends of the leaves with locking devices for the top and bottom chords, so that when closed and locked it acts as a simple span. For a detailed description of the structure with illustrations, the reader is referred to *Engineering News*, Vol. 73, page 108.

The number of bascules constructed in America has become so large that a complete list of them would be beyond the scope of this chapter. The cities of Chicago and Milwaukee have been their largest users in this country, and the Sanitary District of Chicago has built a great number of them over the Drainage Canal. Bascules have also been adopted at Cleveland, Buffalo, Toledo, Peoria, Portland, Ore., Providence, Philadelphia, and other cities. In general, the modern bascule has given good service. For spans requiring leaves not much longer than one hundred feet it is eminently satisfactory; but beyond that limit the first cost of the structure begins to become too high as compared with the vertical lift type of movable bridge, which type is treated at length in the next chapter.

In concluding this chapter the author desires to express his hearty thanks to the following gentlemen and companies for their courtesy in furnishing him with the data concerning their types of bascule bridges:

The Scherzer Rolling Lift-Bridge Company.

The Strobel Steel Construction Company.

J. B. Strauss, Esq., C. E.

Thos. E. Brown, Esq., C. E.

Messrs. Page and Shnable.

The engineers of the Bridge Department of the City of Chicago, and especially John Ericson, Esq., C. E., the City Engineer.

CHAPTER XXXI

VERTICAL LIFT BRIDGES

THE history of vertical lift bridges has been thoroughly worked up by Henry Grattan Tyrrell, Esq., C. E., in a paper presented to The University of Toronto Engineering Society, published in *Applied Science* in 1912, and reprinted in pamphlet form by Mr. Tyrrell. It is well worth perusal by anyone who is interested in the evolution of bridge building. Briefly stated, the development of the vertical lift bridge is as follows:

The first one of which there is any record was a thirty-foot span having a lift of six and a half feet, being a portion of a wooden trestle over the Danube River at Vienna. Subsequent to this a number of very short spans with low lifts were constructed in Europe. The first design for a lift of any importance in respect to both span and rise was one submitted in 1850 by Captain W. Moorson of London in a prize competition on plans for a bridge to cross the Rhine at Cologne. It had a lifting span one hundred feet long and about fifty feet wide with a rise of fifty-four feet. The prize was awarded to another competitor. In 1867 a design was made by Oscar Roper of Hamburg for a three-hundred-foot lift span, which could be raised high enough to permit ocean-going vessels to pass beneath, but nothing ever came of it. In 1872 T. E. Laing proposed a lift span in a bridge over the River Tees at Newport near Middlesbrough, England; but it did not materialize. The movable span was to be two hundred feet long, the lift forty feet, and the maximum vertical clearance ninety feet. It was to be operated by adding and withdrawing water, the tank therefor being a part of the counterweight. In 1878 there was an elaborate design for a lift bridge made by M. H. Matthyssens for a crossing of the Scheldt at Antwerp, involving a span of one hundred and thirty-one feet and about the same clear height. About this time a few small spans with low lifts, generally over canals, were built in various parts of Europe, but until quite lately no vertical lift bridge of any importance has been constructed in Europe.

In 1872 Squire Whipple, one of the pioneers in American bridge building, began to design and build short lift spans with small rises to cross the canals of New York State, including one at Syracuse in which only the deck moves. During the next two decades a number of small vertical lifts were built across canals in the Eastern States, and a few were constructed abroad. In 1892 the author proposed for a crossing of the ship canal at Duluth a vertical lift span of two hundred and fifty feet with a vertical clearance of one hundred and forty feet. As explained in Chapter XXVIII, his design was accepted in competition; but the War Department prevented the building of the structure. A similar bridge of one hundred and thirty feet span and one hundred and fifty-five feet vertical clearance was proposed a few months later by him for a crossing of the South Chicago River at South Halsted Street, Chicago. His proposition was accepted and the bridge was built. A full description of the structure is given in a paper by the author published in the *Transactions* of the American Society of Civil Engineers for January, 1895, and from it the following condensation was made and published in *De Pontibus:*

"The bridge consists of a single, Pratt-truss, through span of 130 ft. in seven equal panels, and having a truss depth of 23 ft. between centres of chord pins, so supported and constructed as to permit of being lifted vertically to a height of 155 ft. clear above mean low water. At its lowest position the clearance is about 15 ft., which is sufficient for the passage of tugs when their smokestacks are lowered. The span differs from ordinary bridges only in having provisions for attaching the sustaining and hoisting cables, guide-rollers, etc., and in the inclination of the end posts, which are battered slightly, so as to bring their upper ends at the proper distance from the tower columns, and their lower ends in the required position on the piers.

"At each side of the river is a strong, thoroughly braced, steel tower, about 217 ft. high from the water to the top of the housing, exclusive of the flag-poles, carrying at its top four built-up steel and cast-iron sheaves, 12 ft. in diameter, which turn on 12 in. axles. Over these sheaves pass the 1½ in. steel-wire ropes (32 in all), which sustain the span. These ropes are double, i.e., two of them are brought together where the span is suspended, and the ends are fastened by clamps, while, where they attach to the counterweights, they form a loop, which passes around a 15-in. wheel or pulley that acts as an equalizer in case the two adjacent ropes tend to stretch unequally.

"The counterweights, which are intended just to balance the weight of the span, consist of a number of horizontal cast-iron blocks about 10 x 12 inches in section, and 8 feet 7 inches long, strung on adjustable wrought-iron rods that are attached to the ends of rockers, at the middle of each of which is inserted the 15-inch equalizing wheel or pulley previously mentioned.

"The counterweights run up and down in guide-frames built of 3-inch angles.

"The weight of the cables is counterbalanced by that of wrought-iron chains, one end of each chain being attached to the span and the other end to the counterweights, so that, whatever may be the elevation of the span, there will always be the same combined weight of sustaining cables and chain on one side of each main sheave as there is upon its other side.

"Between the tops of the opposite towers pass two shallow girders thoroughly sway-braced to each other, and riveted rigidly to the said towers. The main function of these girders is to hold the tops of the towers in correct position; but incidentally they serve to support the idlers of the operating ropes and to afford a footwalk from tower to tower for the use of the bridge-tender. Adjustable pedestals under the rear legs of each tower provide for unequal settlement of the piers which support the tower columns. Each of these pedestals has an octagonal forged steel shaft, expanding into a sphere at one end, and into a cylinder with screw-threads at the other. The ball end works in a spherical socket on a pedestal, and the screw end works in a female screw in a casting which is very firmly attached to the bottom of the tower-leg. It is evident that by turning the octagonal shaft the rear column will be lengthened or shortened. The turning is accomplished by means of a special bar of great strength, which fits closely to the octagon at one end, and to the other end of which can be connected a block and tackle if necessary.

These screw adjustments were useful in erecting the structure, but it is quite likely that they will never again be needed. But in case there is ever any tower adjustment required, it will be found that the extra money spent on them will have been well expended.

"Each tower consists of two vertical legs, against which the roller-guides on the trusses bear, and two inclined rear legs. These legs are thoroughly braced together on all four faces of the tower; and at each tier thereof there is a system of horizontal sway-bracing, which will prevent most effectively every tendency to distort the tower by torsion.

"At the tops of the towers there are four hydraulic buffers that are capable of bringing the span to rest, without jar, from its greatest velocity, which was assumed to be 4 feet per second; and there are four more of these buffers attached beneath the span, one at each corner, to serve the same purpose.

"The span with all that it carries weighs about 290 tons, and the counterweights weigh, as nearly as may be, the same. As the cables and their counterbalancing chains weigh fully 20 tons, the total weight of the moving mass is almost exactly 600 tons.

"Should the span and the counterweights become out of balance on account of a greater or less amount of moisture, snow, dirt, etc., in and on the pavement and sidewalks, it can be adjusted by letting water into and out of ballast-tanks located beneath the floor; and, should this adjustment be insufficient, provision is made for adding small weights to the counterweights, or for placing such weights on the span.

"As the counterweights thus balance the weight of the span, all the work which the machinery has to do is to overcome the friction, bend the wire ropes, and raise or lower any small unbalanced load that there may be. It has been designed, however, to lift a considerable load of passengers in case of necessity, although the structure is not intended for this purpose, and should never be so used to any great extent.

"The span is steadied while in motion by rollers at the tops and bottoms of the trusses. There are both transverse and longitudinal rollers, the former not touching the columns, unless there is sufficient wind-pressure to bring them to a bearing. The longitudinal rollers, though, are attached to springs, which press them against the columns at all times, and take up the expansion and contraction of the trusses. With the rollers removed, the bridge swings free of the columns; and, since the attachments are purposely made weak, the result of a vessel's striking the bridge with its hull will be to tear them away and swing the span to one side. Should the rigging of the vessel, however, strike the span, the effect will be simply to break off the masts without injury to the bridge. This latter accident has happened once already, the result being exactly what the author had predicted. There is a special apparatus, consisting of a heavy square timber set on edge, trimmed on the rear to fit into a steel channel which rivets to the cantilever brackets of the sidewalk, and faced with a 6 x 6-inch heavy angle-iron, to act as a cutting edge. This detail is a very effective one for destroying the masts and rigging of colliding vessels.

"The bridge is designed to carry a double-track street railway, vehicles, and foot-passengers. It has a clear roadway of 34 feet between the counterweight guides in the towers, the narrowest part of the structure, and two cantilevered sidewalks, each 7 feet in the clear, the distance between central planes of trusses being 40 feet, and the extreme width of suspended span 57 feet, except at the end panels, where it is increased gradually to 63 feet. The roadway is covered with a wooden block pavement 34 feet wide between guard-rails resting on a 4-inch pine floor, that in turn is supported by wooden shims which are bolted to 15-inch I-beam stringers, spaced about 3 feet 3 inches from centre to centre. These stringers rivet up to the webs of the floor-beams, and beneath them run diagonal angles, which rivet to the bottom flange of each stringer, and thus form a very efficient lower lateral system. The sidewalks are covered with 2-inch pine planks, resting on 3 x 12-inch pine joints spaced about 2 feet from centre to centre.

"The span is suspended at each of the four upper corners of the trusses by eight steel cables, which take hold of a pin by means of cast-steel clamps. This pin passes through two hanger-plates which project above the truss, and are riveted very effectively to the end post by means of the portal plate-girder strut on the inside and a special, short, cantilever girder on the outside.

"Each portal-girder carries near each end an iron-bound oak block to take up the blow from the hydraulic buffer, which hangs from the overhead girder between towers. Similar oak blocks are let into and project from the copings of the main piers to take up the blow from the hydraulic buffers that are attached to the span.

"The ballast-tanks before alluded to, of which there are four in all, are built of steel plates properly stiffened, and have a capacity of about 19,000 pounds, which is probably more than enough to set the bridge in motion, if it were all an unbalanced load. These tanks serve a double purpose, the first being simply to balance the bridge when it gets out of adjustment because of the varying load of moisture, etc., on the span, and the second being to provide a quick and efficient means of raising and lowering the span in case of a total breakdown of the machinery. If, for instance—which is highly improbable—the operating ropes were broken and had to be detached from their drums, by emptying all of the water out of the tanks the span could be made to rise. It could be lowered again by filling them from a reservoir which is placed on top of one of the towers and kept filled with water at all times by means of a pump in the machinery-house. The water in all of these tanks can be kept from freezing, or the ice therein can be thawed at any time, by turning on steam from the machinery-room into the coils of pipe which they contain.

"The operating machinery is located in a room 37 x 53 feet, the opposite sides being parallel, but the adjacent sides being oblique to each other, the obliquity amounting to about 12 degrees. The placing of this machinery beneath the street was really forced upon the author, who had originally contemplated using electrical machinery and putting it in a house in one of the towers.

"The arrangement of the operating machinery is as follows: Two 70-H.P. steam-engines communicate power to an 8-inch horizontal shaft carrying two 6-foot spiral-grooved, cast-iron drums, around which the ⅞-inch steel-wire operating cables pass. As one of the lifting-ropes passes off the drum, the corresponding lowering-rope takes its place, and vice versa, the extreme horizontal travel being a little less than 12-inches. Thus by turning the drums in one direction the span is raised, and by turning them in the other direction the counterweights are raised, and the span consequently is lowered. When the span is at its lowest position, the full power of one engine can be turned on to pull up on the counterweights, thus throwing some dead load on the pedestals of the span, after which the drums can be locked. Before the bridge was completed the writer considered that this would be necessary, in order to check vibration from rapidly passing vehicles; but such has not proved to be the case, for the span is very rigid, and the amount of the vibration is not worth mentioning. It is possible, though, that in some other lift-bridges, where the ratio of live load to dead load is greater, this feature of operation could not be ignored.

"The engines are provided with friction-brakes that are always in action, except when the throttle is opened to move the span; consequently no unexpected movement of the span is possible.

"The raising-ropes, after leaving the drums, pass out of the machinery-house to and beneath some 5-foot idlers under the towers, thence up to the top of the north tower, where they pass over some 4-foot idlers and the main 12-foot sheaves. Four of them here pass down to the north end of the span, and the other four run across to the other tower over more idlers, then down to the south end of the span.

"The lowering-ropes, after leaving the drums in the machinery-room, pass under some idlers below the north tower, and thence up to more idlers at the top of the tower. Four of them here pass down to the counterweights in the north tower, and the other four run across, over intermediate idlers in the overhead bracing, to the main 12-foot sheaves of the south tower, then downward to the counterweights.

"In addition to the previously mentioned method of moving the span by the water-ballast, there is a man-power operating apparatus of simple design in the machinery-house, which, when used alone, can raise and lower the span slowly in case the steam-power gives out, or more rapidly when combined with the water-ballast method.

"As the span nears its highest and lowest positions, an automatic cut-off apparatus in the machinery-room shuts off the steam from the cylinders and thus prevents the hydraulic buffers from being overtaxed."

In Fig. 31*a* is presented a view of the Halsted Street Lift Bridge partially raised. The original design called for the use of two sixty-five horse-power electric motors, but the city of Chicago required a steam engine plant of one hundred and fifteen horse-power instead. The cost of this plant for both operation and maintenance was found to be excessive; and in 1907 electric motors were substituted for the steam engines. Operation by steam had required the services of three engine men, two signal men, four policemen, and one coal shoveler, ten men in all, their combined wages amounting to one thousand dollars per month; and in addition there were one hundred and seventy dollars per month expended for coal, as it was necessary to keep the boilers going at all times during the season of navigation. The cost of the electric power for intermittent service proved to be only one hundred and fifty dollars per month; and the services of only one tender were required, while two had been formerly needed with steam. The change resulted in a saving of over three thousand dollars per annum in the operating expenses.

In the before-mentioned paper published by the American Society of Civil Engineers there appeared the following:

"If the author were to design another lift-bridge similar to the Halsted Street structure, and if he were given *carte blanche* in the designing, he would make the following improvements:

"1. Curve the rear columns and arch the overhead girders at tops of towers, so as to improve the general appearance.

"2. Operate by electricity instead of by steam.

"3. Place the machinery-house in one of the towers and dispense with the operating-house on the span, letting the operator stand in a bow-window of the machinery house so as to command a view of the river in both directions.

"4. Omit the water-tanks as an unnecessary precaution, and rely on the great capacity of the electric motors to overcome any temporary unbalanced load.

"5. A simpler and less expensive adjustment at feet of rear columns.

"6. Cast steel instead of cast iron for all machinery.

"7. Catch the balancing chains in buckets placed on top of the span instead of hanging them to the counterweights."

In the later designs for vertical lift bridges prepared by his firm, the author was persuaded, rather against his will, to omit the hydraulic buffers and the balancing chains, on the plea that with electric power these are not necessary; but after an experience of several years with lifts in which these two features of his first design were omitted, he has decided to adopt them again in some of his future vertical lift bridges.

In large and heavy lift-spans the unbalanced load of the cables augments materially the starting torque and adds considerably to the amount of power used per annum, besides increasing somewhat the first cost of the machinery. Whether it is good policy in any particular case to adopt the counter-balancing chains is to be determined by an economic study. If the annual interest on the difference between the first cost of the said chains and the saving in cost of machinery is greater than the value of

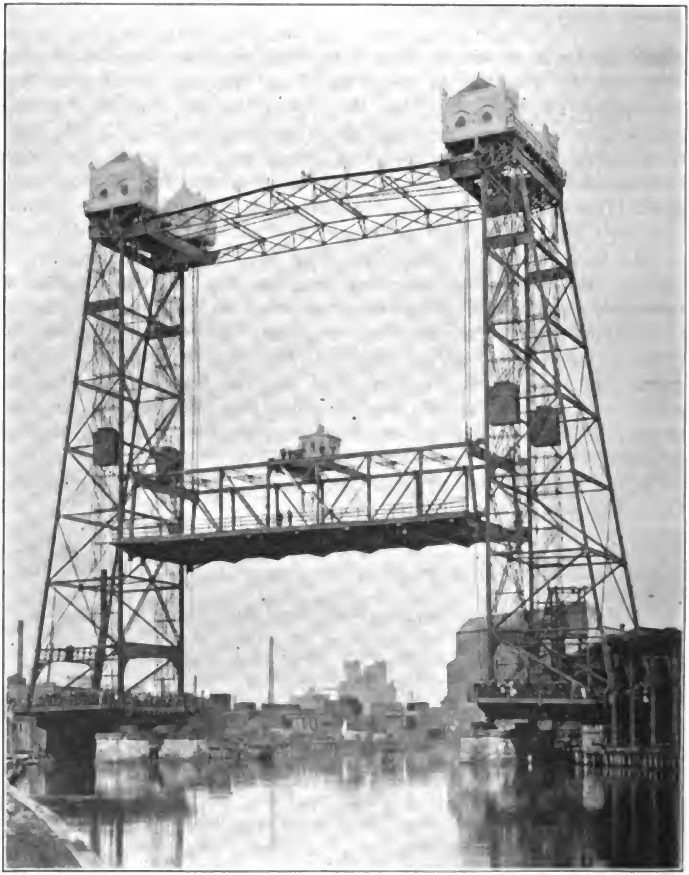


FIG. 31a. Halsted Street Lift Bridge over the Chicago River at Chicago, Ill.

the annual saving of power which they effect, they should be omitted, but otherwise they should be used. All depends upon the question of how often the bridge is to be operated. If it is to be opened only a few times per day, the additional expense would be unwarranted; but if it is to be used very often, the saving of cost of power may be great. In the Pacific Highway Bridge, designed by the author's firm and now under construction, the economic study indicated a stand-off, hence the chains were omitted, as it was desirable to keep down the initial cost of structure to a minimum; but all the Chicago vertical lift-spans should have been provided with the chains, as the number of openings often runs as high as seventy-five per day. Moreover, in some cases a flat rate based upon the peak load is charged for the electric power—and in such cases it is evident that the adoption of the detail for balancing the weight of the cables would be in the line of true economy.

Good and effective hydraulic buffers that are properly maintained in commission are a wise precaution against accident; and they certainly relieve most effectively all jar in bringing the span to rest.

For many years after the completion of the Halsted Street structure the author endeavored unsuccessfully to build similar bridges at other places, the main reason for his failures being that he often ran into political and financial conditions of such a nature that his engineer's conscience prevented his dealing with the parties interested.

In 1894 he made plans for a bridge over the Missouri River at Kansas City, known at first as the Winner Bridge, in which there was a span of four hundred and twenty-five feet carrying a lifting deck; but the construction thereof was delayed for many years. The original design was described in *De Pontibus;* but it was changed materially when the bridge was built some eight years ago, mainly because of the developments that had taken place in bridge designing in the preceding decade. A description of how the structure was actually built will follow presently.

From 1894 until 1907 no progress worth mentioning was made in the building of vertical lift bridges, mainly for the reason that the author's patents prevented other engineers from entering the field, and because of his personal discouragement previously mentioned. But soon after the formation of the firm of Waddell and Harrington in 1907, the author heard from good authority that the changes made in the machinery of the Halsted Street Bridge had converted it into the most satisfactorily operating movable bridge in Chicago; hence he and his partner made a joint study of how to improve on the design of the Chicago bridge; and soon there came to them a request from F. W. Fratt, Esq., C. E., the new president of the Union Depot, Bridge, and Terminal Railway Company, to make a study and estimate of cost for finishing the partially constructed Winner Bridge, which his company had bought in, upon the general lines described in *De Pontibus.* They did so, making a number of changes in the old design, the principal of which were the following:

First. Adopting riveted construction instead of pin-connected.

Second. Telescoping the hangers inside of the vertical posts of the supporting trusses instead of letting them pass outside.

Third. Using concrete instead of cast-iron counterweights and placing them at the ends of the span instead of at the panel points.

Fourth. Operating from a machinery house at each end of the span instead of from a single house at mid-span, and using wire ropes instead of shafting for the transmission of power.

Mr. Harrington's extended experience in various lines of mechanical engineering, especially that obtained as engineer to the C. W. Hunt Company of New York, enabled the firm to effect many valuable improvements in operation, not only in this structure, but also in other vertical lift bridges built later.

While Mr. Fratt and his clients were debating about the advisability of undertaking the work of building the structure, the firm was retained to rebuild the Iowa Central Railway Company's bridge across the Mississippi River at Keithsburg, Ill. Bids were obtained upon both a swing and a vertical lift, showing a material economy for the latter, which was, consequently, adopted and built. The span is two hundred and thirty-four feet and the maximum vertical clearance fifty-five feet. It carries a single-track railway only. In its design there is an innovation which results prove was not a good one. The operating house is placed at one end of the movable span instead of at the middle. It was so located in order to reduce the dead load moment on the span, especially as the machinery is unusually heavy on account of the operation being by gasoline engines. Such a location was a violation of a principle of æsthetics, viz., that perfect symmetry in a layout is the acme of artistic designing; and the result showed that it was not good practice, because, on account of the inequality in length of the operating ropes, the stretches therein were unequal, necessitating frequent adjustments, the neglect of which caused a jerky motion of the span when being raised or lowered. The defect is of but little importance, nevertheless its cause should always be avoided in future construction.

During the building of the Keithsburg Bridge, a little highway lift at Sand Point, Idaho, was designed and constructed. It showed great economy as compared with a swing span.

Next came the Hawthorne Avenue Bridge over the Willamette River at Portland, Oregon, with a lift span of two hundred and forty-four feet and a vertical clearance of one hundred and thirty-five feet, carrying a double-track street railway, two wagonways, and two footwalks. Two views of this structure are shown in Figs. 31*b* and 31*c*. It is of the same general type as the Halsted Street Bridge, except that there is no overhead span between tops of towers.

While this structure was under way Mr. Fratt and his associates, after

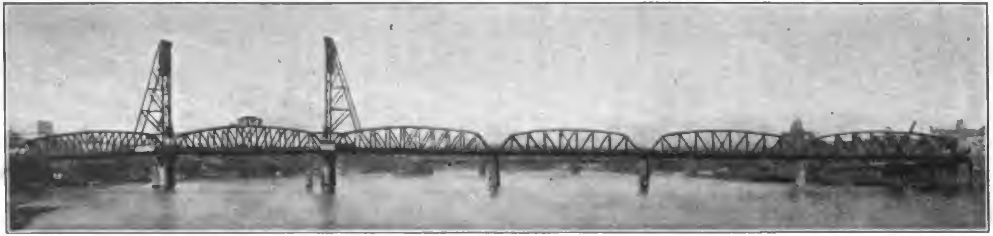


FIG. 31*b*. Hawthorne Avenue Bridge over the Willamette River at Portland, Ore.—Lifting Span Down.

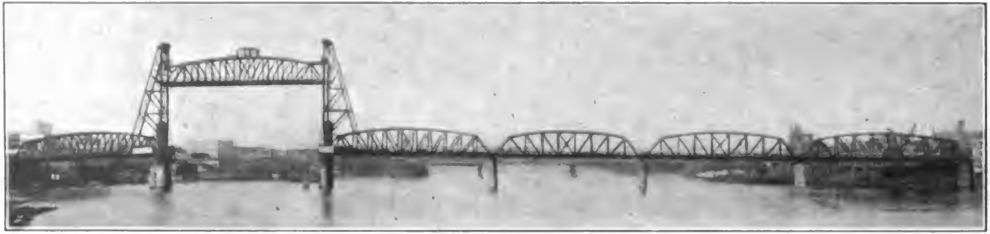


FIG. 31*c*. Hawthorne Avenue Bridge over the Willamette River at Portland, Ore.—Lifting Span Up.

long deliberation, decided to build their bridge; but before they would make up their minds to adopt the lifting deck, they had a large working model made of it to scale and operated by electric power; and although this worked to perfection, they still were not satisfied until they had an expert committee of civil and mechanical engineers examine the plans, specifications, and model and report upon the efficiency and practicability of the design. This committee was composed of the following well-known gentlemen: Thos. E. Brown, Esq., C.E., of New York City; S. B. Fisher, Esq., C.E., of St. Louis; Prof. W. V. M. Goss, of the University of Illinois; and Geo. W. Jackson, Esq., of Chicago. This committee gave its unanimous approval to the project, and the bridge was built. The following is a description of the structure, which is shown with the deck down in Fig. 31*d* and with the deck up in Fig. 31*e*.

The bridge proper, *i. e.*, the portion between the established harbor lines, and excluding the approaches, consists of three double-deck, riveted-truss spans, providing on the lower deck two standard railway tracks, and on the upper deck two street car tracks and separate roadways and sidewalks for vehicular traffic and pedestrians. To permit the passage of boats, one of the three spans contains a lifting deck, which consists of a double-track railway bridge floor, the metal thereof being nickel steel so as to reduce the weight to be lifted, with a lateral system that includes special wind chords, all supported by stiff hangers, also of nickel steel, from each panel of the upper trusses. When the deck is in its lowest position, a pin in the end of each hanger rests on a socket in diaphragms in the post above, transmitting the live load directly to the upper trusses. Each hanger is arranged to telescope into the truss post above, and is attached to two cables which pass up and over a sheave on the top of the truss, thence to the end of the span and over a common drum at one corner thereof, and thence downward to a counterweight. There is, thus, a separate counterweight for each hanger. Operation is effected by rotating the four drums at the upper corners of the span. The two drums at each end are on a single shaft which is geared down to a motor. In order to synchronize mechanically the movements of the machinery at opposite ends of the span, a double rope drive is provided connecting the two sets of machinery. A full-size model of this drive was made and tested by the engineers before the design was adopted, in order to satisfy the projectors of the enterprise that it would work satisfactorily. The counterweights for the rope drives are arranged so that one rope is taut for driving in each direction. Under ordinary operation both motors are in service; but, should one motor fail, the entire deck can be handled by the motor at the opposite end through this rope drive.

When the deck reaches its lowest position, locks automatically engage each hanger and the ends of the deck. All locks are withdrawn by one operation by means of a motor and gears in the south machinery house.

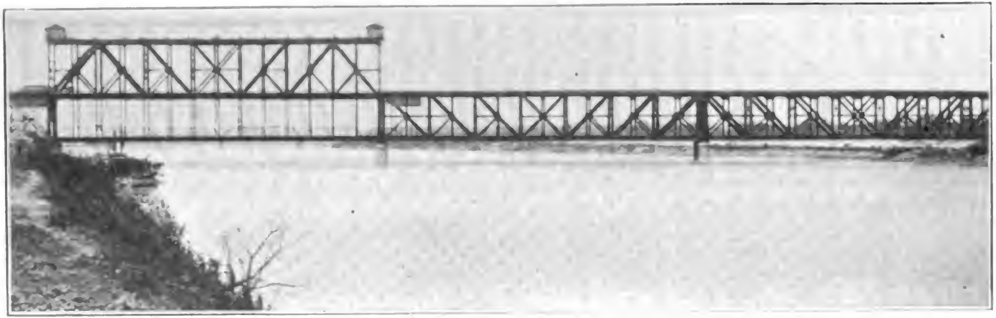


FIG. 31*d*. Fratt Bridge over the Missouri River at Kansas City, Mo.—Lifting Deck Down.

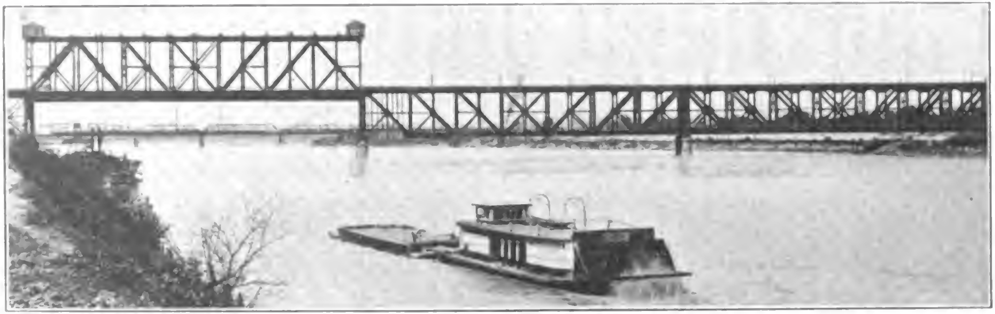


FIG. 31*e*. Fratt Bridge over the Missouri River at Kansas City, Mo.—Lifting Deck Up.

In addition to the manifest advantage of maintaining traffic on the upper deck at all times regardless of the movement of boats, this movable deck afforded large economies in construction, and it will afford also similar economies in operation.

The vertical clearance when the deck is raised to its full height is fifty-five feet above high water, and the horizontal clearance for vessels is four hundred and thirteen feet, the overhead through span and the two adjoining deck spans being each four hundred and twenty-five feet long. The reason for the excessively great horizontal clearance is that the superstructure is built on old piers that had been standing in the river for nearly two decades.

The lifting deck, which weighs one and a half million pounds, is fully balanced by the counterweights and is always locked down when in service. It is raised to its full height or is lowered in fifty seconds by electric power. The total cost of the bridge and its approaches was $2,200,000. The total weight of metal was over eighteen thousand tons, and a number of the pieces handled weighed over one hundred tons each. There were some twenty-five miles of rivet holes reamed in the field and about half a million field rivets were driven. The amount of paint used on the metalwork was fifty tons.

The next structure containing a lift span built by the author's firm was the Arkansas River Bridge between the cities of Fort Smith and Van Buren, Arkansas. As can be seen from Figs. 31*f* and 31*g*, it contains nine spans all alike, one of them being lifted so as to give the usual vertical clearance requirement of about fifty feet. It carries a railway, street railway, and vehicular and pedestrian traffic. The distinctive feature of this structure is that it is arranged so that should ever the channel shift permanently, the towers and machinery can be taken down, moved, and re-erected so as to lift any of the other spans.

Next came the highway bridge at Tehama, California, a comparatively small structure containing no special features; and this was followed by the little M. L. and T. bridge, Figs. 31*h* and 31*i*, which has been adopted as a standard for its small bayou crossings by the Southern Pacific Railway Company. It is operated by one man, as it is not opened often.

Next in order came the great Oregon-Washington Railroad & Navigation Company's bridge, at Portland, Oregon, carrying traffic just like that of the Fratt Bridge, but with the difference that the overhead span, instead of being fixed, was made movable so as to permit the passage beneath of the largest ocean-going vessels. Like the Fratt Bridge, the main portion of the structure consists of three spans, but the total length of them is only seven hundred and ninety-six feet, that of the movable one being two hundred and twenty feet.

In Figs. 31*j*, 31*k*, and 31*l* is shown the structure in its three principal positions, viz.: first, with both the movable span and the movable deck

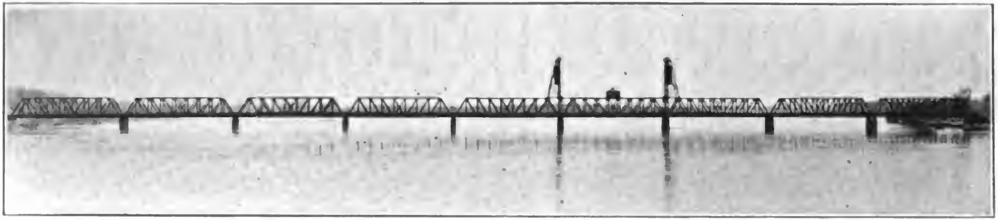


FIG. 31*f*. Arkansas River Bridge between Fort Smith and Van Buren, Ark.—Lifting Span Down.

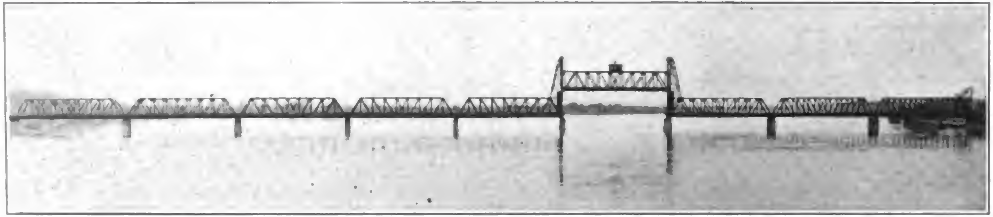


FIG. 31*g*. Arkansas River Bridge between Fort Smith and Van Buren, Ark.—Lifting Span Up.

down and taking care of the traffic above and below; second, with the movable span in commission for highway traffic, but with the deck beneath raised to its full height so as to permit the passage of small steamers; and third, with both the movable span and the movable deck raised to their greatest height so as to provide for the passage of the tallest-masted ocean-going vessels that enter the port. Fig. 31*m* has been added, as it gives an excellent view of almost the entire structure, approaches included. Attention is called to the height of the water, the photograph having been taken at its maximum stage.

The distinguishing features of the structure are its unusually heavy construction and the method of lifting the two decks of the movable span either together or separately. The upper deck was designed to carry the heaviest possible city traffic, including pedestrians, electric railway cars, road-rollers, and lorries; and the lower deck to care for the heaviest locomotives and cars used on the Harriman system. The movable span is a through one for highway traffic, while the two flanking spans are through for railway traffic and deck for highway traffic. The lift span rests on columns placed on the piers. The lower deck has a clearance of twenty-six feet above low water and one of only five feet above high water, the base of rail being six feet higher. The upper deck is fifty-two and a half feet above the lower deck. The latter has a separate lift of forty-six feet, making a clear height of seventy-two feet above low water, or fifty-one feet above high water, without moving the upper deck. The latter has a lift of ninety-three feet, so that when hoisted with the lower deck also in raised position, the total lift of the lower deck is one hundred and thirty-nine feet, and the total clearance is one hundred and sixty-five feet above low water and one hundred and forty-four feet above high water. This clears the highest-masted vessels entering Portland. When the lower deck alone is lifted, all but the largest steamboats plying the river can pass at ordinary stages of water. The vertical lift span is much the heaviest of that type thus far built, the total load lifted, including counterweights, amounting to nearly nine million pounds. The towers are about two hundred and seventy feet high above low water.

The main or upper deck is lifted at each corner by sixteen steel cables, two and a quarter inches in diameter, passing over a sheave fourteen feet in diameter. Each sheave weighs twenty-four tons; but as the boxes were attached before hoisting, the weight to be lifted was thirty-five tons. These main sheaves rest on heavy sheave girders between the tower posts.

In each tower there is a single main counterweight made of concrete weighing over one million seven hundred thousand pounds, the over-all dimensions being forty feet height, thirty feet width, and ten feet thickness. These counterweights were constructed in place around a steel framework. At the corners are projecting guides that engage the fixed guides on the tower. The lower deck has separate counterweights that were cast in sections on the main deck and after hardening were trans-

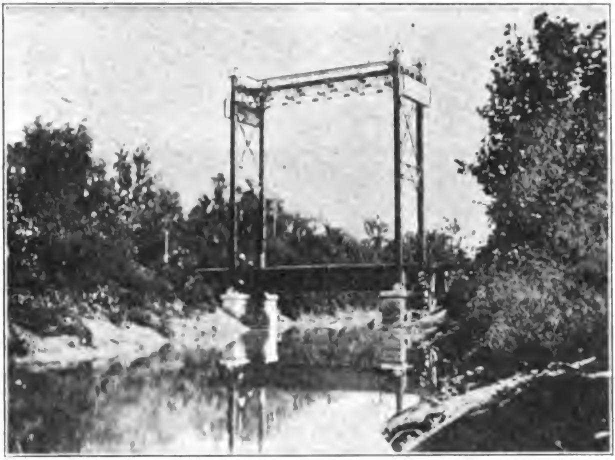


FIG. 31*h*. Vertical Lift Bridge over the Big Choctaw Bayou, Louisiana, on the Line of the M. L. and T. R. R. & S. S. Co.—Span Down.

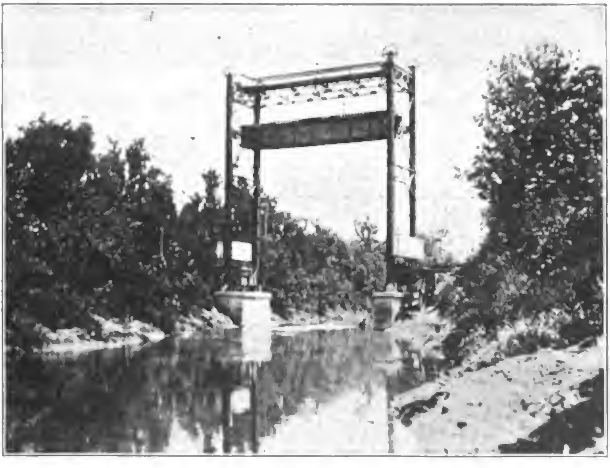


FIG. 31*i*. Vertical Lift Bridge over the Big Choctaw Bayou, Louisiana, on the Line of the M. L. and T. R. R. & S. S. Co.—Span Up.

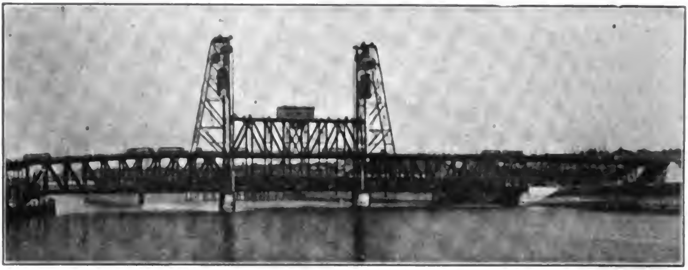


FIG. 31*j*. O.-W. R. R. and N. Co.'s Bridge over the Willamette River at Portland, Ore.—Lifting Deck and Lifting Span Down.

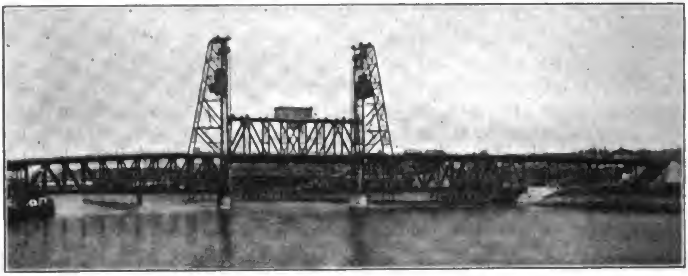


FIG. 31*k*. O.-W. R. R. and N. Co.'s Bridge over the Willamette River at Portland, Ore.—Lifting Deck Up.

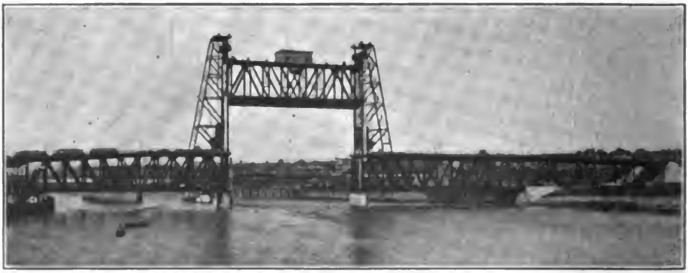


FIG. 31*l*. O.-W. R. R. and N. Co.'s Bridge over the Willamette River at Portland, Ore.—Lifting Deck and Lifting Span Up.

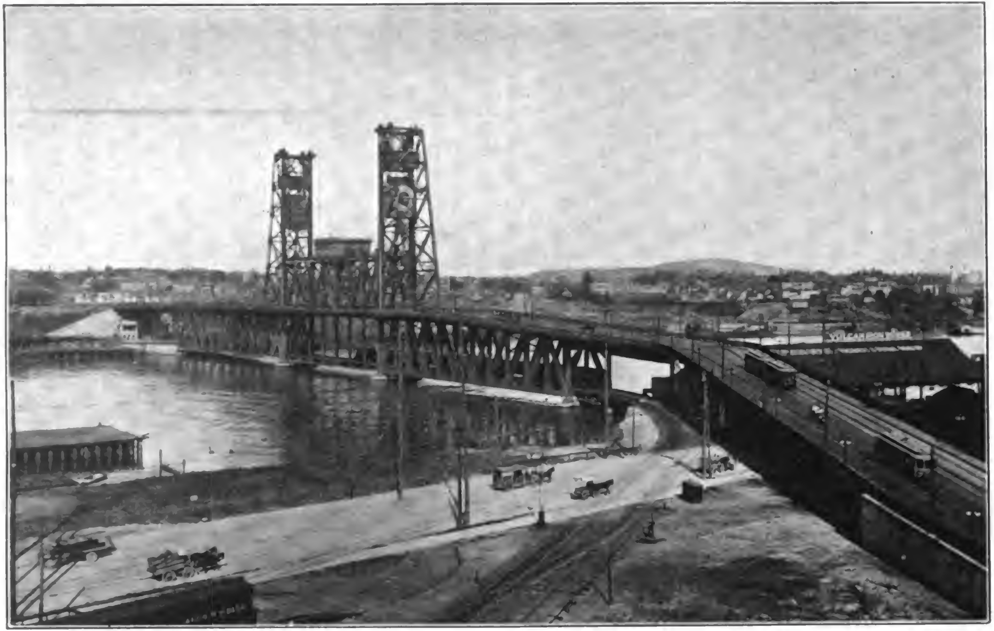


FIG. 31*m*. O.-W. R. R. and N. Co.'s Bridge over the Willamette River at Portland, Ore., with the Approaches.

ported to the towers and raised into position. In order to keep the proper adjustment between span and counterweights there were provided a large number of concrete blocks one cubic foot in size. These can be added to the counterweights as required. The total weight of the lower deck and its attachments is over one million pounds, which, of course, is also that of the balancing counterweights.

The operating machinery is placed in a house on the top of the movable span and at its mid-length, covering the full width between trusses. The operator's room is suspended beneath this house, so that he can observe the main deck traffic as well as the river traffic. The machinery for operating the lower deck is driven by two electric motors of two hundred horse-power each, placed on the down-stream side of the house, and that for the upper deck by two similar motors located on the upstream side thereof. In the operator's room is placed the mechanism for locking and unlocking both the lifting span and the lifting deck. The cams for holding down the lower deck lock automatically; but they are unlocked by a special mechanism driven by a small motor. The main deck also locks automatically, but it is released by the operator's turning a wheel.

The erection of the superstructure was a most formidable task, because the channel had to be kept open at all times for the passage of boats, including high-masted sailing vessels. The immense weight of the movable span, the fact that it was to rest on columns high above the water, and the swiftness of the current made it seem to the contractor too difficult to build the span on scows and float it into place, as was done on the Hawthorne Avenue lift bridge over the same river some two miles distant where the conditions were less onerous. It was, therefore, necessary to erect the movable span at its full height, supporting it by four wooden Howe trusses. The total cost of the structure was $1,700,000.

Figs. 31*n* and 31*o* are photographic views of the City Waterway Bridge in the city of Tacoma, Wash. Its peculiar features are the unusually great height of the deck above the water and the overhead span for carrying water pipes. It will be noticed that the structure is on a grade. The Puyallup River Bridge located only a few blocks away is quite similar in type but of shorter span and narrower roadway.

Fig. 31*p* is a reproduction of a photograph of the Pennsylvania Railroad Bridge over the South Branch of the Chicago River in the city of Chicago. It is built on quite a skew, necessitating vertical rear legs in the towers. The length of span is two hundred and seventy-two feet. It is a double-track structure, and some three hundred trains cross it daily. It is opened on the average about seventy-five times per day during most of the navigation season. This structure is designed for a possible future 24-foot raising of the grade line.

Fig. 31*q* is a view of another Pennsylvania Railroad Bridge, crossing the Calumet River in South Chicago. Strictly speaking, there are two bridges, one being located close alongside the other, and each carrying

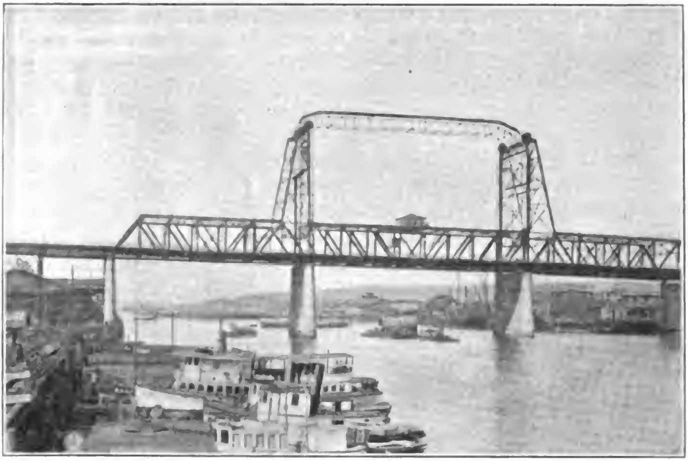


FIG. 31*n*. Bridge over the City Waterway at Tacoma, Wash.—Lifting Span Down.

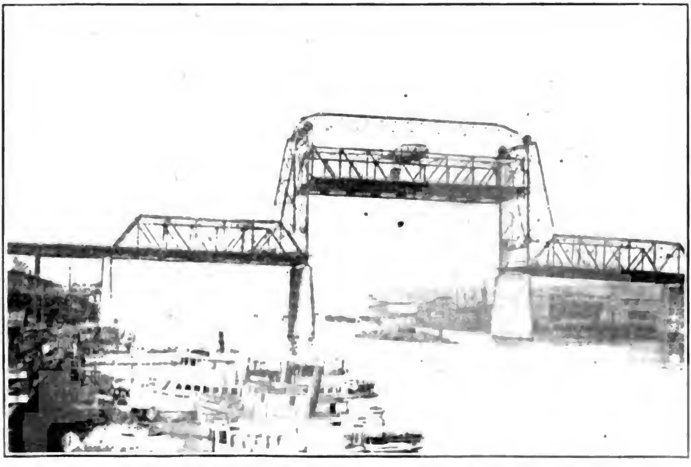


FIG. 31*o*. Bridge over the City Waterway at Tacoma, Wash.—Lifting Span Up.

a double track. The skew is about the same as in the bridge last described, but the span length is only two hundred and ten feet. In the rear of the left-hand tower will be seen a Strauss bascule bridge in open position.

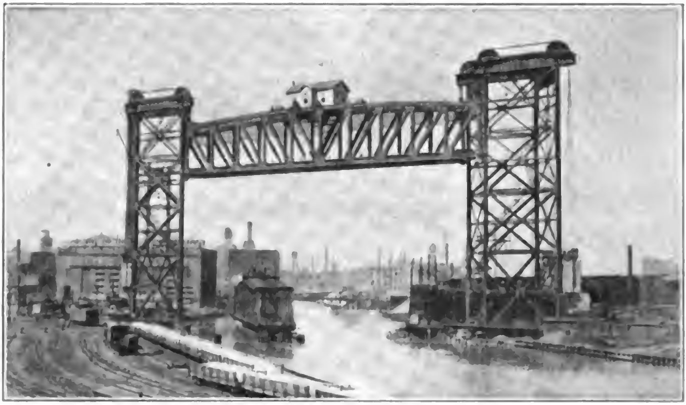


FIG. 31*p*. Pennsylvania R. R. Co.'s Bridge over the South Branch of the Chicago River, Chicago, Ill.

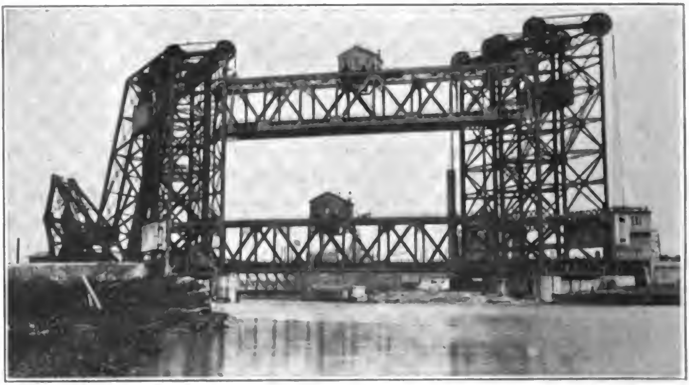


FIG. 31*q*. Pennsylvania R. R. Co.'s Bridge over the Calumet River, South Chicago, Ill.

In *Engineering News*, Vol. 68, p. 1056, will be found a description of the Columbia River Bridge at Trail, British Columbia, together with the layout and a photographic view of the structure. Its peculiarity is that the towers and the machinery have been temporarily omitted, as at present there is no steamboat traffic on the river at that place; but the complete plans are drawn for the said towers and machinery, and the steelwork is all arranged even to the open rivet-holes for attaching the new construction at any time in the future so as to pick up either of the two intermediate spans.

The Lake Shore and Michigan Southern Railway Bridge over the Calumet River in South Chicago is very similar to the Pennsylvania Railroad Company's Bridge for the same crossing, described previously. At first the L. S. & M. S. Co. intended to build a four-track lift span, and the plans were prepared accordingly; but later they decided to follow the lead of the P. R. R. Co. and build two bridges close together, the object being to provide for a possible break-down. Their tracks at each end are so arranged that the traffic can be switched from either bridge to the other.

Figs. 31*r* and 31*s* illustrate the Yellowstone River Bridge on the line of the Great Northern Railway. It is located a short distance above the junction of that river with the Missouri in Montana, and there is a similar structure over the latter a few miles away. These lift spans have lengths, respectively, of two hundred and seventy-five and three hundred feet. The Missouri River lift-span is the longest yet built, and in fact the clear opening is the largest in the world for opening bridges, barring only the Fratt Bridge at Kansas City, where, as explained previously, the determination of opening was fixed in advance by the existing piers of an unfinished structure. It does not seem logical for the War Department to require such large openings in Missouri River bridges near the head of navigation thereon, while much smaller openings have been permitted everywhere else below; but such was the case, and the railroad company and their consulting engineers could do naught else but comply with the law.

Figs. 31*t* and 31*u* show the Black River Bridge on the line of the Louisiana and Arkansas Railway in Arkansas. Its lift span is one hundred and sixty-three feet long, and the vertical clearance is the usual fifty feet. Figs. 31*v* and 31*w* illustrate the bridge over the Little River on the same line of railway in the same state. Its movable span is one hundred and eighteen feet long. Attention is called to the symmetry of the layout for this structure and to the dolphins employed for protecting the piers.

Figs. 31*x* and 31*y* show a photograph of the Salem, Falls City, and Western Railway Bridge over the Willamette River at Salem, Oregon. The length of the movable span is one hundred and thirty-one feet. In one view the span is shown rising as a steamer is approaching; and the picture indicates how close to a vertical lift it is permissible to run a vessel before raising is begun. In some of the Chicago bridges which have to be opened often and over which pass daily many trains, the steamers are allowed to come very close, indeed, to the structure before hoisting is

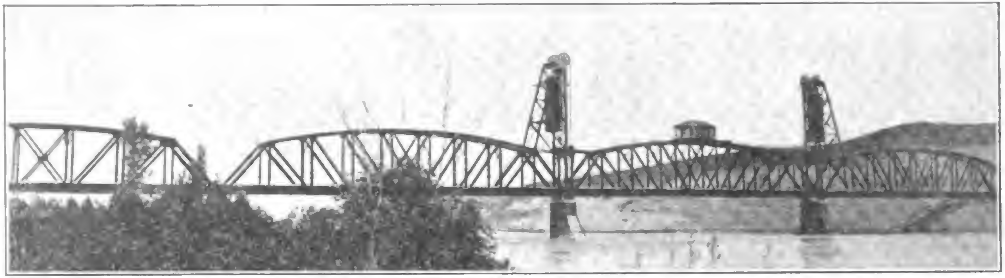


FIG. 31*r*. Great Northern Railway Bridge over the Yellowstone River, Montana.—Lifting Span Down.

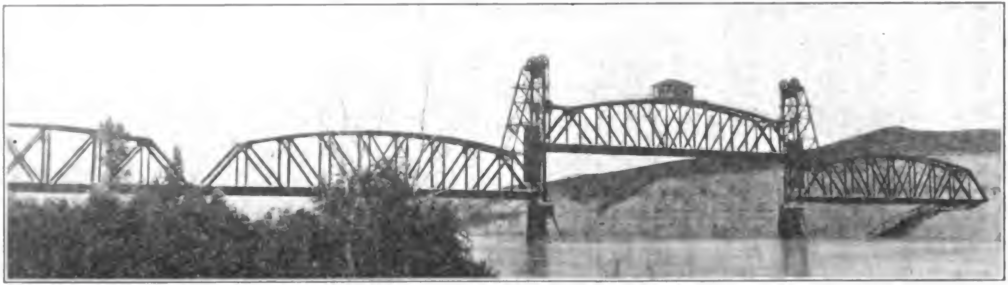


FIG. 31*s*. Great Northern Railway Bridge over the Yellowstone River, Montana.—Lifting Span Up.

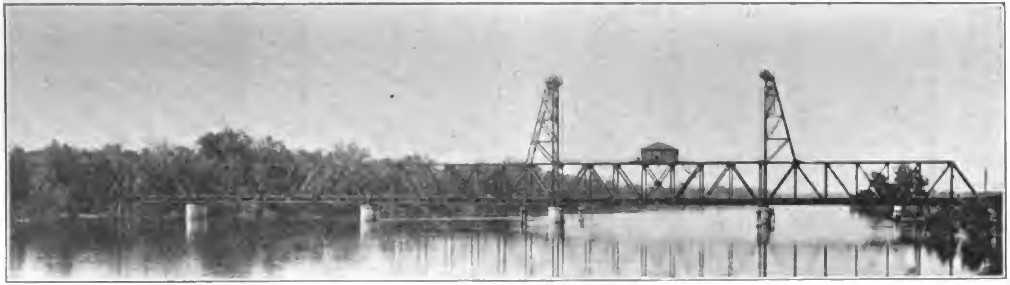


FIG. 31*l*. Louisiana & Arkansas Railway Bridge over the Black River in Louisiana.—Lifting Span Down.

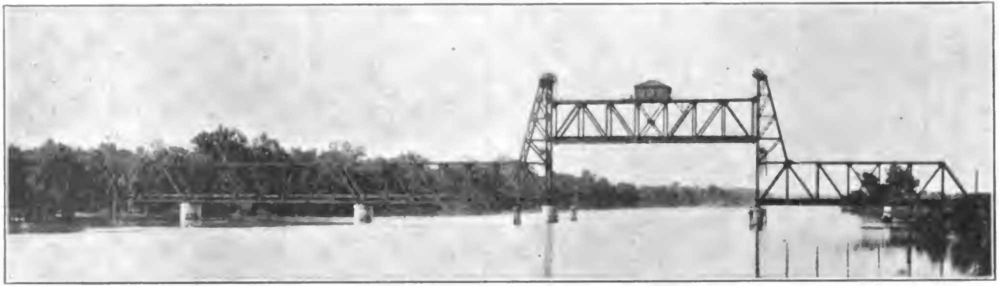


FIG. 31*u*. Louisiana & Arkansas Railway Bridge over the Black River in Louisiana.—Lifting Span Up.

started; and the lowering is begun before the vessel has actually passed the bridge tangent.

In Fig. 31*z* is given a profile of a bridge over the Don River at Rostoff, Russia, the movable span, towers, and machinery of which were designed by the author's firm, the flanking spans having been designed by the bridge engineers of the Russian Government. The length of the moving span is two hundred and ten feet and that of each flanking span three hundred and seventy-seven feet. Attention is called to the unusually great curvature of the rear legs of the towers, adopted so as to conform to the decided curvature that exists in the top chords of the flank-

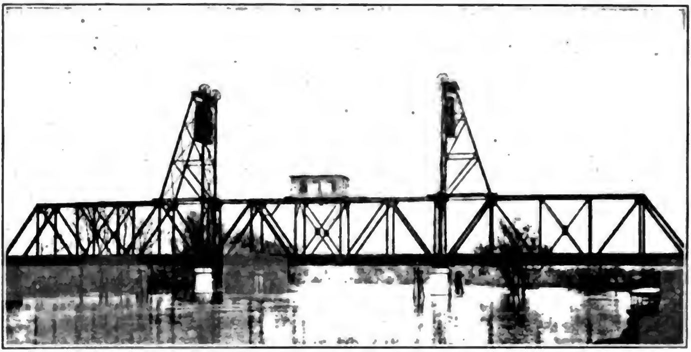


FIG. 31*v*. Louisiana & Arkansas Railway Bridge over Little River in Louisiana.—Lifting Span Down.

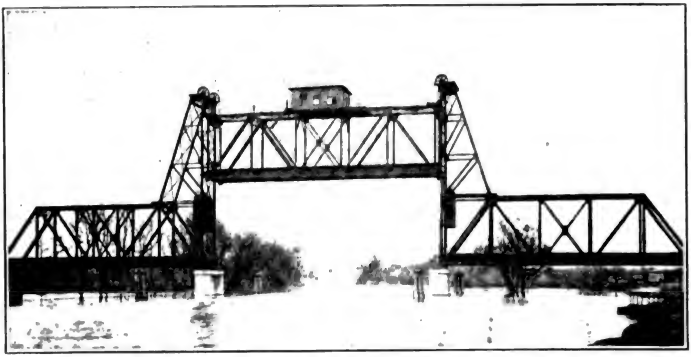


FIG. 31*w*. Louisiana & Arkansas Railway Bridge over the Little River in Louisiana.—Lifting Span Up.

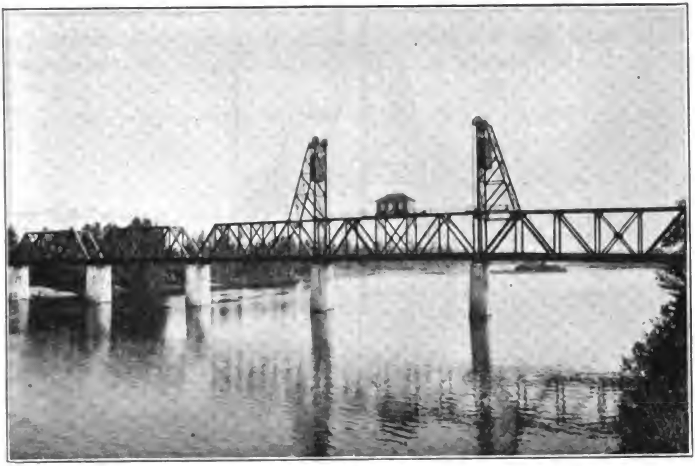


FIG. 31*x*. Salem, Falls City, and Western Railway Bridge over the Willamette River at Salem, Oregon.—Lifting Span Down.

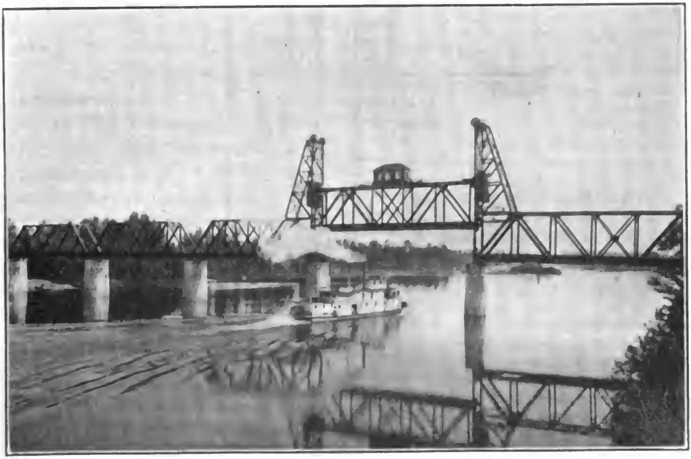


FIG. 31*y*. Salem, Falls City, and Western Railway Bridge over the Willamette River at Salem, Oregon.—Lifting Span Up.

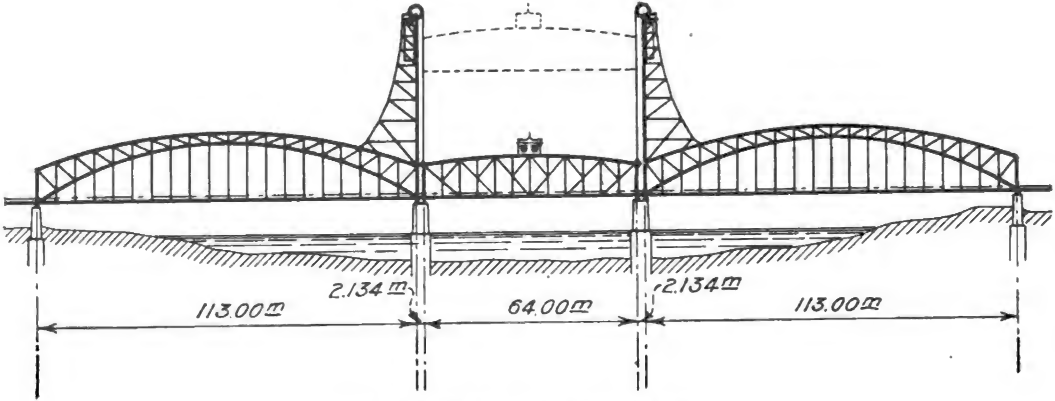


FIG. 312.—Don River Bridge at Rostoff, Russia.

ing spans. This structure is now under construction. It is to be operated by electricity.

In Fig. 31*aa* there is shown a portion of the long, deck, plate-girder bridge over the North Thompson River near Kamloops, B. C., with its lifting towers and machinery, all of which can be shifted at any time so as to pick up any one of the numerous spans that were made alike mainly for this purpose.

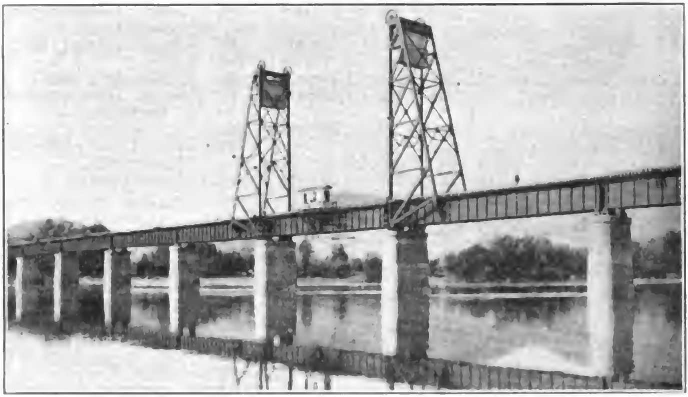


FIG. 31*aa*. Canadian Northern Pacific Railway Bridge over the North Thompson River in British Columbia.

In Fig. 31*bb* is illustrated a half-through, plate-girder bridge over the Oromocto River on the line of the St. John and Quebec Railway in New Brunswick, with its lifting span and towers, which it will be noticed are of a different type from those shown in Fig. 31*aa*, because of there being no necessity for rear columns and bracing.

In addition to the types of vertical lift bridges covered by the patents of the author and those of his firm, both Mr. Strauss and Mr. Rall have lately patented vertical lifts operating like their patented bascules but lifting at both ends of the span instead of at one. These are certainly more expensive than the vertical lifts herein described, as was proved in one case in the author's practice by detailed comparative estimates made by his firm's computing force.

In the following table are given, as nearly as may be in chronological order, the various vertical lift spans, designed and engineered by the author and his firm, together with their general dimensions, load to raise and lower, and height raised, including four small ones designed by contractors and checked by the firm's computers.

Bridge	Distance c to c of Trusses in Feet	Length of Lift Span in Feet	Load to be Lifted in Pounds	Height to be Lifted in Feet	Remarks
1. Halsted St.	40	130	600,000	140	
2. Keithsburg	17.7	229	940,000	48	Spans Interchangeable for Lifting
3. Sand Point	18	83	60,000	50	Small Highway Bridge
4. Hawthorne Ave.	23.3	244	1,770,000	116	
5. Mo. River, K. C.	32	425	1,560,000	43	Lifting Deck Only
6. Arkansas River	24.5	192	1,500,000	50	Spans Interchangeable for Lifting
7. Tehama	21.8	167	266,000	63	
8. M. L. & T. Ry.	16.3	50	69,000	43	Plate-Girder Lift-Span
9. O.W.R.R.&N.—U.D.	34	211	3,420,000	89	Upper Deck
O.W.R.R.&N.—L.D.	34	211	1,060,000	46-89	Lower Deck
10. City Waterway at Tacoma	53.3	214	1,640,000	78	Carries Water Pipe Overhead
11. Puyallup River at Tacoma	43.3	161	943,000	115	Carries Water Pipe Overhead
12. Penn. No. 443	31.3	210	1,837,000	101	Two Bridges Like This Close Alongside
13. Trail	21	171	266,000	50	Towers and Machinery Omitted Temporarily
14. Little River	17.3	118	380,000	44	
15. Black River	17.3	165	620,000	56	
16. St. Francis River	17.3	162	620,000	73	
17. Ill. River	18.3	173	698,000	43	
18. Red River of North	19.2	140	120,000	25	Light Highway Span
19. Penn. No. 458	29.5	273	3,006,000	114	To be Duplicated in Future
20. Harrisburg	17	200	656,000	63	
21. Salem	17.5	131	462,000	54	
22. C.N.P.R.R.No.10	18.2*	90	236,000	56	Plate-Girder Lift-Span
23. St. Paul	21.8	189	850,000	56	
24. L.S.&M.S.R.R.No.6	31	210	1,410,000	101	Two Bridges Like This Close Alongside
25. International Falls	15	75	200,000	50	Light Highway Span
26. Mo.Riv.G.N.Ry.	17.7	296	1,560,000	43	Spans Interchangeable for Lifting
27. Grand Rapids	19	83	78,600	30	Light Highway Span
28. Yellowstone	17.7	271	1,370,000	43	Spans Interchangeable for Lifting
29. Oslo, Minn.	18	155	112,000	25	Light Highway Span
30. Oromocto	17.5	58	147,000	59	Plate-Girder Lift-Span
31. Don River	30.6	210	1,600,000	131	Under Construction
32. Caddo Lake	18.5	92	218,000	53	Small Highway Bridge
33. Pacific Highway	41	272	2,400,000	139	Under Construction

* Width of towers.

The vertical lift bridges thus far constructed as listed above may be divided into three general types, viz.:

A. Those in which the whole span is raised.

B. Those in which a deck only is raised up to an overhead fixed span.

C. Those in which a deck is raised to an overhead movable span, which also can be raised to clear high-masted vessels.

Class A may be subdivided into the following groups:

a. Those structures in which there is an overhead span.

b. Those structures in which there is no overhead span.

Those in Group "*a*" may be still further divided thus:

Alpha. Where the supports consist of four columns with trusses between their tops, and

Beta. Where the supports consist of towers braced on four faces.

Those in Group "*b*" also may be subdivided thus:

Gamma. Where the rear columns of the towers are inclined and where there is a main sheave at each of the four corners, and

Delta. Where the rear columns of the towers are vertical and where there are eight sheaves in all, one over each of the four columns of each tower.

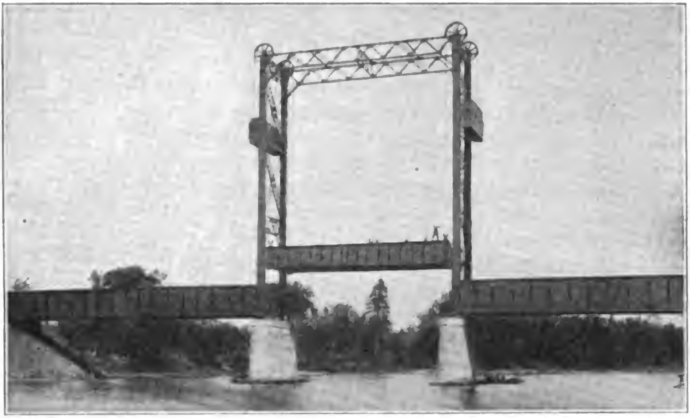


FIG. 31*bb*. St. John and Quebec Railway Bridge over the Oromocto River in New Brunswick.

The Halsted Street Bridge (Fig. 31*a*) and the Hawthorne Avenue Bridge (Figs. 31*b* and 31*c*) represent Class A; the Fratt Bridge (Figs. 31*d* and 31*e*) illustrates Class B; and the Oregon-Washington Railway and Navigation Company's Bridge (Figs. 31*j*, 31*k*, 31*l*, and 31*m*) is an example of Class C.

Group "*a*" is represented by the Tacoma City Waterway Bridge (Figs. 31*n* and 31*o*) and by the M. L. & T. (Southern Pacific) Railway Bridge (Figs. 31*h* and 31*i*), and Group "*b*" by the Hawthorne Avenue Bridge (Figs. 31*b* and 31*c*), and the Fort Smith-Van Buren Bridge (Figs. 31*f* and 31*g*).

The Alpha subdivision is exemplified by the M. L. & T. Ry. Bridge (Figs. 31*h* and 31*i*), Beta by the Halsted Street Lift Bridge (Fig. 31*a*), Gamma by the Rostoff Bridge over the River Don in Russia (Fig. 31*z*), and Delta by the two Pennsylvania Railroad bridges in Chicago (Figs. 31*p* and 31*q*).

Before drawing this chapter to a close it is necessary to sound a note of warning to the computer who makes the calculations for a lift-span that has cantilever brackets, in relation to the effect of a live load on one cantilever only. In an ordinary span of this type the uplift at the corner due to the overturning moment of the live load on the bracketed portion is resisted by the dead load reaction there; but in the case of the lift-span there is no such reaction; consequently, there is nothing to resist the said overturning effect except the unbalanced load of the cables (if any), the starting friction of the sheave-journals, and the holding-down power of the operating ropes and bridge locks. For ordinary cases where only narrow sidewalks are cantilevered from a wide deck, this overturning moment may be ignored; but where either the street railway tracks or the wagonways are cantilevered, as in the case of the Hawthorne Avenue Bridge, some effective means of resisting this overturning moment must be provided. . In that case cantilever brackets from the substructure were put on so as to furnish at their end bearings for receiving the extremities of the end cantilever brackets of the lifting span.

The true economy of the vertical lift bridge as compared with both the swing span and the bascule is proved beyond the peradventure of a doubt by the thirty or more structures listed and described above. The type has come to stay; and it will continue to be used more and more as time goes on; for not only is it inexpensive in first cost, comparatively speaking, but it is also simple, rigid, easy to operate, and economical of power. It has met with considerable opposition up to the present time, mainly from the owners of bascule patents; but it has overcome that opposition most satisfactorily and unequivocally, consequently the future of the type may be counted upon as assured.

CHAPTER XXXII

RIVETED VERSUS PIN-CONNECTED TRUSSES

For more than a quarter of a century there has been a desultory controversy in the technical press concerning the relative merits of riveted and pin-connected bridges. At first the division of opinion was between English and American engineers, but later the dissension took place among the engineers of the United States. In spite of all the discussion that was evolved, no conclusion was reached as to which of the two types was preferable; and, truth to tell, in the old days both types were so exceedingly faulty that, when one now looks back over the controversy, he is forced to the conclusion that both sides were wrong. Time has finally settled the question by showing that each type has its place in good construction, riveted trusses being preferable for short spans and pin-connected trusses for long ones; but the dividing span length appears to have been and still is a changing quantity—increasing as time goes on. Twenty-five years ago it was about one hundred (100) feet, while today a consensus of opinion of the best authorities would probably place it between two hundred and fifty (250) and three hundred and fifty (350) feet. The author has built several riveted spans of unusually heavy construction, as long as four hundred and twenty-five (425) feet, and there are today several still longer ones contemplated and even under way.

The disadvantages of the old type of riveted spans were as follows:

First. They were lattice or multiple-intersection bridges, involving on that account considerable ambiguity of stress distribution.

Second. Their connections were exceedingly clumsy, crude, unscientific, and ofttimes weak.

Third. The weights of metal required for their manufacture were excessive.

Fourth. The secondary stresses involved were great; but as very little was known in those days concerning their importance, they were ignored.

Fifth. Where there was a possibility of washout during erection, the riveted structures involved more risk of disaster than did the pin-connected ones, because of the longer time required to couple up and make safe.

The advantages of this old type were:

First. The deflections and vibrations were comparatively small.

Second. In case of the derailment of a train upon a riveted-lattice span its chance of escaping collapse was greater than that of a corresponding pin-connected span under like conditions.

The disadvantages of the old type of pin-connected spans were:

First. They were light and vibratory.

Second. Their connections were unscientific and often weak.

The advantages of this old type were:

First. The central pin-connections generally avoided ambiguity of stress in truss members.

Second. The weights of metal involved in their manufacture were comparatively small—in fact, generally too small for good construction and satisfactory operation.

Third. The secondary stresses were usually low, owing to the freedom for motion at the ends of most of the main members.

Fourth. The time required for erection was comparatively small.

The modern riveted truss, when used for spans of ordinary length, involves practically all of the good points of both the old riveted and the old pin-connected trusses with but few of their disadvantages. In appearance it is similar to the old pin-connected truss, from which it was evolved by substituting stiff members for eyebars and riveted connections for pins (excepting only that, at the end panel points of the bottom chord, pins are employed to connect the truss as a whole to the pedestals). This type of truss involves no ambiguity of primary stress distribution except in the mid-panel of trusses having an odd number of panels, where, of necessity, two intersecting stiff diagonals are employed; and in that case the ambiguity is of minor importance, being confined to the said two stiff diagonals. The appearance of this truss is good—just as good as and more substantial looking than that of a corresponding pin-connected truss. The weight of metal which it requires is but little, if any, greater than that needed for a similar pin-connected truss. The deflections are smaller than those of either of the two old types of truss herein referred to, and the vibrations are as small as it is practicable to make them by any system of trussing. The chance of successfully resisting disaster from a derailed train is far better than that for a corresponding pin-connected structure, and nearly as good as that for one of the old lattice girders. But, unfortunately, the secondary stresses still remain large, although far smaller than those in the old lattice girders with their eccentric connections. Such secondary stresses as remain are due to the rigidity of the connections and not to faulty intersections nor to unbalanced sectional areas of main members.

Although modern riveted truss bridges often weigh a little more than the corresponding pin-connected truss bridges, the pound price for their manufacture is a trifle smaller; but this is offset more or less by the fact that the erection of riveted structures is somewhat more expensive than that of similar pin-connected ones. On the whole, there is today for ordinary spans but little difference in total cost of completed bridges between riveted and pin-connected structures.

As for the possibility of loss of structure from wash-out during erection, the modern riveted truss bridge has so few panels and connections, and the use of pneumatic riveters has so reduced the time required for riveting that the danger involved by adopting riveted instead of pin connections is a matter which can generally be ignored.

Rigidity is such an important feature in modern railroad bridges with their heavy and rapidly passing live loads that the leading bridge engineers of America now adopt riveted trusses almost invariably for short spans and for those of moderate length. The longer the span the greater is the economy of both metal and cost of erection for the pin-connected type of construction, especially when the secondary stresses are given due consideration. The author believes that for present conditions it is better, generally, to limit the length of span for riveted truss bridges to about four hundred and fifty (450) feet, due cognizance being taken of the fact that the heavier the structure the shorter should be the limiting span length for riveted construction.

There is one advantage that a riveted truss bridge has to an eminent degree over a pin-connected one, viz., that in case it be carried off its piers by flood, it stands a good chance of avoiding absolute destruction. A pin-connected structure under such conditions is sure to become a mass of tangled wreckage that will cost more to remove than its scrap value; while a riveted structure may survive the shock well enough to be used again with moderate repairs. During the Kaw River flood of 1903, all but one of some twenty bridges near Kansas City were carried away, and all but two of those that went out were total wrecks. Those two were riveted truss bridges, one an old one of rather unscientific design and the other a first-class modern structure. The old bridge was cut apart and rebuilt, but the resultant structure proved to be about as expensive as a new bridge, while the spans of the new one were picked up, placed upon the masonry, and submitted for several days to train traffic without any repairs. Of course, there were a few broken pieces in each span, but these were of such minor importance that they were either repaired or replaced without stopping the traffic. Considering the fact that one of the spans was transported seven hundred and fifty feet down stream and rolled over and that the other two spans were also carried some distance by the current, their resurrection in such good shape speaks well for the strength and stiffness of the riveted-truss type of bridge. Fig. 1*k* illustrates the bridge in question.

In pin-connected bridges there is undoubtedly a tendency to wear in the joints. This has been shown by numerous old, light, short-span bridges and elevated railroads that have been taken down, and by rust-wash superposed upon the paint below the pins on eyebars of bottom chords. For light and short spans this wear is an important factor, as it limits the life of the structure to a short term of years; but in heavy and long spans its action is so exceedingly slow that it might require centuries to effect any real damage. Of course, the slight elongation of the eyes in the eyebars will gradually lessen the camber, but that is of no serious importance. It is only when the metal back of the pin is too much reduced for safety or when the pin is materially cut into that the damage due to such wear becomes serious.

CHAPTER XXXIII

DIMENSIONING FOR CAMBER

CAMBER was introduced into bridges originally for the sake of appearance. If a long line which is practically horizontal sags a trifle, the result is not only unæsthetic, but even suggests a lack of strength; while if it rises slightly, the effect is graceful. From this standpoint, it would be best to construct all spans having an approximately horizontal line as a salient feature with such camber that under the worst conditions this line would still have a slight upward curvature. Camber is also introduced into bridges today for the purpose of reducing or eliminating certain secondary stresses, as was explained in Chapter XI.

Camber is sometimes used to meet requirements of under-clearance. It is often necessary that no portion of a bridge lie below a certain horizontal plane. By cambering trusses with nominally straight bottom chords, the end panels of each span can be brought down close to the clearance line, without the centre panels sagging below it when the span is loaded. However, this result can be obtained more easily by raising the structure a trifle; and the point will not be considered further in this chapter.

In plate-girder spans the principal horizontal line is generally the bottom of the girders, although sometimes the floor line is more prominent. Girder spans are so short that there is little need of cambering them for the sake of appearance or for any other reason.

In through truss bridges the principal horizontal line is practically always the bottom chord, although sometimes the floor line is conspicuous. Camber is provided in practically all simple truss spans. Since the tops of the stringers must generally be parallel to the bottom chords, the introduction of the camber into the structure will cause the floor to rise slightly in the centre portion of each span when the bridge is unloaded. This is unobjectionable in a single span; but when a bridge contains several spans, it causes a series of humps in the floor which are rather undesirable. These humps are not usually so noticeable in a highway bridge, as the amount of camber under full dead load need not be very great, and the eye will not notice it very much on a solid floor; but in railway bridges a larger camber will be required under dead load, and the rails make the humps quite conspicuous. Most railways specify that either half or all of the camber under dead load shall be taken out of the floor by dapping the ties, unless so doing would cut into the timber too deeply. The half-dapping is preferable in most respects, as it does not cut into the ties so much, and as it will make the grade practically straight when a line of passenger cars is on a span. The full-dapping scheme has the advantage that the track men can level up the track more easily, as the grade is straight when there is no live load on the bridge. Both plans are somewhat objectionable, because it is quite difficult to dap the ties properly in either case, and because the replacement of the deeply dapped ties is very troublesome. In fact, some engineers consider that when a railway company requires that half or all of the camber in the floor is to be taken out by dapping the ties, it is better to secure this result by reducing the camber in the steelwork and keeping the ties of uniform depth. This plan is certainly much simpler and more satisfactory, so far as the track is concerned; and where the question of appearance is not very important, as is often the case, it offers the best solution of the problem.

In deck truss spans the bottom chord is usually the most important horizontal line. As the top chord is almost always parallel to it, humps similar to those produced in the floors of through bridges by camber will occur in the floors of deck spans as well. They can be handled as was indicated for the case of through spans, or else the top chord can be given a smaller camber than the bottom chord.

In cantilever bridges the grade line will ordinarily be straight when there are several spans; but it generally rises to an apex at the centre of the suspended span, when there is only a single main opening. The bottom chord of the suspended span is generally parallel to the grade, but it may be somewhat arched. The bottom chords of the cantilever and anchor arms may be parallel to the floor line, or they may be arched, in which case the top chord may be at the floor line, or it may extend high above it. When the floor and the chords are arched, any further cambering for the sake of appearance is unnecessary. The secondary stresses will generally have to be considered. If the floor line and the chords be nominally straight, it will be necessary to camber in order to prevent the horizontal lines from sagging under any probable conditions. Here also consideration of the secondary stresses will generally be necessary.

Steel arch spans usually have no horizontal lines other than the floor line. When there is but a single span, the floor is generally laid out to a slight curve with its highest point at the centre of the span, as this adds greatly to the appearance of such a structure; but where there are several spans in succession, the grade should be straight, or else it should rise but a trifle at the centre of each span. The cambering must be such that the floor line will never sag below the horizontal.

The only nominally horizontal lines of a suspension bridge are the floor and the chords of the stiffening trusses, which lines are nearly always parallel. These lines are made somewhat arched, so that it will be unnecessary to camber for the sake of appearance. Cambering for secondary stresses in stiffening trusses of suspension bridges is generally unnecessary.

The problem of cambering a swing span is, in general, one of cambering two simple spans, each of a length equal to that of one arm of the draw, with certain special features due to the fact that the two spans are continuous.

A vertical lift span should be cambered in the same manner as a simple span of the same type. The cambering of the towers requires special consideration, however. The cambering of a bascule will depend upon its shape. A single-leaf bascule can generally be cambered like a simple span, while the cambering of a double-leaf bascule will usually be similar to that of either a simple span or an arch, depending on its form.

Reinforced concrete bridges should be constructed so that the floor line will be practically straight under dead load. It is frequently advisable to have the grade rise a trifle at the centre of each span, so as to ensure that a slight settlement of the falsework will not cause it to sag. The bottom lines of reinforced concrete girders should always be arched a trifle, if possible; and if made nominally straight, they should be cambered somewhat, so that they will not sag below a straight line, if the falsework should settle a little. The appearance of a concrete structure is always quite important; and sagging lines may ruin the beauty of an otherwise fine-looking bridge.

THE CAMBERING OF PLATE GIRDERS

As was previously stated, there is generally no need of cambering plate girders. In case it is desired to do so, however, the girders should bend at the splices only and be straight between them. If a girder be cambered so that it will be straight when loaded with full dead plus live plus impact loads, it will be necessary to use a camber of about one one-thousandth ($^1/_{1000}$) of the span length when the ratio of length to depth of girder is ten (10). However, it is hardly worth while to camber quite this much, because such a load will be rare. Ordinarily, for girders having the above ratio of length to depth, a camber of one-twelve hundredth ($^1/_{1200}$) of the span length, or one (1) inch per hundred (100) feet, will be about right; and for any other ratio of length to depth, the camber should be $\frac{l}{12{,}000\,d}$, where l is the span length and d the depth of the girder, both values being expressed in the same unit. The angle of bending, ϕ, at each splice will be given with sufficient accuracy by the formula,

$$\phi = \frac{l}{1500\,d\,(n+1)}, \qquad \text{[Eq. 1]}$$

where n is the number of splices, and ϕ is measured in radians.

If the ratio $\frac{l}{d}$ is 10, this becomes

$$\phi = \frac{1}{150\ (n + 1)}. \qquad \text{[Eq. 2]}$$

If the end stiffeners be set at right angles to the flanges, they will be inclined to the vertical at an angle ϕ' (in radians) given by the expression,

$$\phi' = \frac{n\ l}{3000\ d\ (n + 1)}. \qquad \text{[Eq. 3]}$$

If $\frac{l}{d} = 10$, this becomes

$$\phi' = \frac{n}{300\ (n + 1)}. \qquad \text{[Eq. 4]}$$

The girders are bent at the splices only, in order to simplify the shop work. Each piece of the web can then be put through the multiple-punch as usual. The top and the bottom flange angles will differ only in that one rivet space at each splice will be larger in the top than in the bottom flange. In the vertical splice plates, the lines of rivets on opposite sides of the splice will make an angle with each other equal to the angle of bend. As the over-all lengths of the top and the bottom flanges will be different, the bottom lateral system (if any) will vary slightly from the top lateral system.

THE CAMBERING OF SIMPLE TRUSS SPANS

Theoretically, a truss should be so cambered that under full dead plus live plus impact loads the bottom chord will be straight. However, the provision of a camber of this amount accentuates the humps in the grade of the floor, or increases the dapping of ties to avoid them. Furthermore, since the occurrence of this maximum loading is rare, there is no need of providing for it, so far as the question of the appearance is concerned. It is also unnecessary to consider it as acting when it is desired to camber in order to reduce the secondary stresses, as was pointed out in Chapter XI. For truss spans in general, a good rule is to camber for dead plus one-half of the live and impact loads; and this is the amount called for in the specifications of Chapter LXXVIII. A factor a little greater than one-half might be nearer the proper value for single-track railway spans, but the difference is of small importance. The length of a chord under the above loading may be called its "normal" length, as it is the average of the lengths under dead load only and under full load. The normal length of a span is an important figure, as the expansion shoes must be set in accordance with it; hence it is desirable to use this length in all the camber calculations. It might further be noted that the cambers obtained by the use of the above loading are about the same as those obtained by lengthening the top chord members by one-eighth ($\frac{1}{8}$) of an inch for every ten (10) feet of length, which rule has been very widely used for many years.

The camber of trusses in general is to be figured in the following manner: The truss is assumed to have its *nominal* outline—that is, the bottom chord straight and of *nominal* length, and all posts perpendicular to it and of *nominal* length,—when under dead plus one-half of live and impact loads. The corresponding lengths of the diagonals are then figured; and also those of the top chord members, unless the said top chord be straight, in which case they will be of the same lengths as the corresponding bottom chord members. The lengths of the various members under the above conditions will be termed their *normal* lengths. Next, the changes in the lengths of the various members due to the dead load plus one-half of the live and impact loads over the entire span are to be figured, and these amounts added to the *normal* lengths of the compression members, and subtracted from the *normal* lengths of the tension members, thus giving the *theoretic* shop lengths. All of the above lengths should be computed to the nearest hundredth ($\frac{1}{100}$) of an inch. If the truss be a riveted one, the *actual* shop lengths are to be found by expressing the *theoretic* shop lengths to the nearest thirty-second ($\frac{1}{32}$) of an inch, which is as close as the shop work can be done. If the truss be pin-connected, the *actual* shop lengths are to be found by first expressing the *theoretic* shop lengths to the nearest thirty-second ($\frac{1}{32}$) of an inch, and then by further increasing the length of each compression member by one sixty-fourth ($\frac{1}{64}$) of an inch at each end which bears on a pin, and similarly decreasing the length of each tension member by the same amount, in order to allow for the play at the pin-holes.

The camber of a riveted truss in its no-load condition is figured by drawing a Williot diagram (see Chapter XII), using for the changes in the lengths of the various members the differences between their normal lengths and their theoretic shop lengths. The adoption of these differences, rather than those between the normal and the actual shop lengths, saves a little time; and it is really a trifle better, as the truss, if riveted up under these conditions, will take its nominal outline under dead plus one-half of live and impact loads, while if the actual shop lengths were used it would not do so exactly. The employment of the theoretic shop lengths will mean possible mismatching of holes amounting to one sixty-fourth ($\frac{1}{64}$) or even one thirty-second ($\frac{1}{32}$) of an inch, when the connections are reamed to templets, but this is of no importance whatsoever; while if the truss is assembled in the shop and its connections reamed (which is the proper method), even these slight mismatchments will be corrected, because the truss will be laid out for the reaming in accordance with the camber diagram as above computed.

The camber of a pin-connected truss in its no-load condition is also to be figured by means of the Williot diagram, using, however, for the changes in the lengths of the various members the difference between the normal and the actual shop lengths. The adoption of the latter is necessary in this case, as the pins cannot take up slight differences as can the hot rivets. The actual outline of a pin-connected truss when under dead load plus one-half of live and impact loads is likely to vary slightly from the nominal outline, due to the differences between the theoretic and the actual shop lengths, and to the uncertainty as to the amount of play in the pin-holes; but as the amount of this variation in camber caused by such differences in any one member should rarely be as much as one-eighth ($\frac{1}{8}$) of an inch, and as the variations due to different members are likely to be compensating, no account need be taken thereof. It might be well always to handle the differences between theoretic and actual shop lengths so that the actual shop lengths of the compression members will tend to be too great rather than too small, and those of the tension members too small rather than too great. As a result of this procedure the truss will have its nominal outline under a load a trifle greater than the dead load plus one-half of the live and impact loads, which is entirely satisfactory.

For long spans the above-explained method of figuring shop lengths and camber should generally be followed. For shorter spans the approximate method before mentioned—that of increasing the length of each top chord member over that of the corresponding bottom chord member by one-eighth ($\frac{1}{8}$) inch for each ten (10) feet of length—will give good enough results, if the distorting effect of secondary members, such as hip verticals, be duly cared for. In general, it might be said that this approximate rule can be applied to all spans two hundred (200) feet or less in length. Its application to a truss with parallel chords causes the shop lengths of all diagonals and of all posts (c. to c.) to be alike throughout the span, so that it is particularly advantageous for that type of truss. It would be a very good rule to use the approximate method for trusses with parallel chords, and the exact one for those with polygonal top chords.

The approximate method is to be applied to a truss with parallel chords in the following manner: The theoretic normal length of the entire bottom chord (c. to c. of end pins) is known, and from that its theoretic length under no load is figured. The shop lengths of the various panels are then made equal (for the ordinary case, in which the panels are nominally of equal length). These panel lengths should be expressed to the nearest thirty-second ($\frac{1}{32}$) of an inch, and then the actual no-load and normal lengths of the entire bottom chord should be determined. These should rarely vary from their theoretic values by as much as one-eighth ($\frac{1}{8}$) of an inch. The shop length of each top chord panel is next found by making it longer than the shop length of the corresponding bottom chord panel by one-eighth ($\frac{1}{8}$) of an inch for each ten (10) feet of length, the said shop length being always expressed to the nearest thirty-second ($\frac{1}{32}$) of an inch. The shop lengths of the various posts which form a portion of the main truss system are then taken equal to the nominal truss depth, and should likewise be expressed to the nearest thirty-second ($\frac{1}{32}$) of an inch. The shop length of any hanger which carries a single panel load only (as the hip-vertical in a through Pratt truss, or the hangers in a through triangular truss with verticals) is made less than the nominal truss depth by the amount of its extension under dead load plus one-half of live and impact loads; and the shop length of any strut which carries a single panel load only (as the struts in a deck triangular truss with verticals) is similarly made greater than the nominal truss depth. The above shop lengths are also taken to the nearest thirty-second ($\frac{1}{32}$) of an inch. The shop lengths, l_D, of the various diagonals and of the inclined end posts are then figured by the formula,

$$l_D = \sqrt{\left(\frac{l_T + l_B}{2}\right)^2 + d^2}, \qquad \text{[Eq. 5]}$$

in which

l_T = shop length of top chord panel,
l_B = shop length of bottom chord panel,
and d = nominal truss depth.

If there be two intersecting diagonals in the centre panel of a riveted truss that has an odd number of panels, it will be best to make the shop length of each half-diagonal one-half of l_D, and figure on riveting the centre connection after the span has been swung, if this be practicable.

The camber of the truss under no load is then determined in the following manner: The panel points of both chords are first assumed to lie on arcs of circles, with the posts, hangers, and struts radial, the hangers and struts being assumed for the present to have the same length as the posts. The radius R_B for the circular curve containing the bottom chord panel points is given by the formula,

$$R_B = l_B \times \frac{d}{(l_T - l_B)}, \qquad \text{[Eq. 6]}$$

and the radius R_T of the circular curve containing the top chord points by the expression,

$$R_T = l_T \times \frac{d}{(l_T - l_B)}. \qquad \text{[Eq. 7]}$$

From these equations we get

$$R_T - R_B = d, \qquad \text{[Eq. 8]}$$

as is self-evident.

The distance y of the centre point of the circle on which the bottom chord points lie above the line through the end pins is

$$y = \frac{l^2}{8\,R_B} = \frac{l^2\,(l_T - l_B)}{8\,l_B\,d}, \qquad \text{[Eq. 9]}$$

where l is the shop length of the bottom chord.

If we represent the number of panels by n, Equation (9) takes the form,

$$y = \frac{n\,l\,(l_T - l_B)}{8\,d}. \qquad \text{[Eq. 10]}$$

For any panel point distant n' panel lengths from one end of the span, assuming the curve to be a parabola, the camber c is given by the equation,

$$c = \frac{n'\,(n - n')\,l\,(l_T - l_B)}{2\,d\,n} \doteq \frac{n'\,(n - n')\,l_B\,(l_T - l_B)}{2\,d}. \qquad \text{[Eq. 11]}$$

The camber of any top chord point is evidently equal to that of the corresponding bottom chord point. Since each post is radial, the top chord point will not lie vertically over the bottom chord point, but will be located a distance x to one side, the value of x being given by the expression,

$$x = \frac{(n - 2\,n')\,(l_T - l_B)}{2}. \qquad \text{[Eq. 12]}$$

The value of c for any point which is connected to the main truss system by a hanger or strut must now be corrected by the amount that the shop length of the said hanger or strut differs from the nominal depth of the truss.

The "shop lengths" mentioned thus far in the discussion of the approximate method of cambering have referred entirely to the distances between panel points. In a riveted truss, these lengths will be used for the various members directly; but in a pin-connected truss, each compression member must be lengthened one-sixty-fourth ($\frac{1}{64}$) of an inch at each end which bears on a pin, and each tension member must be similarly shortened by an equal amount, in order to take care of the play at the pin-holes.

The approximate method has been explained above entirely with regard to its application to Pratt and single-intersection Warren or triangular trusses. It may also be applied to multiple-intersection trusses, but the use of such types today is so rare that an extended treatment of their cambering is not justified. A Petit truss, even if of short span, should be cambered by the exact method, in order to reduce as far as possible the distorting effects of the secondary members.

The form of the camber curve of a nominally straight bottom chord will be nearly the same, no matter which method of computing the camber is followed. The curve given by the exact method will be a little sharper at the centre and flatter at the ends, so that if the two methods gave the same cambers at the centre in any particular case, panel-points near the quarter-points of the span would be located a trifle higher by the approximate method than by the exact one. This difference might amount to as much as one-quarter (¼) of an inch for a span two hundred (200) feet long. This would mean that if such a truss were cambered by the approximate method, and then loaded so as to take out the entire camber at the centre, the quarter-points would remain about one-quarter (¼) of an inch high, instead of coming down into the straight line between the end-pins. This is, of course, of no practical importance.

The cambering of trusses has thus far been discussed on the assumption that the span would be erected on falsework. It is frequently necessary, however, to erect simple-truss spans by cantilevering out from adjacent spans, or from the banks. The shop lengths of the various members of such a truss can be determined in the same way as for a span which is to be erected in the ordinary manner; but it will be necessary to figure the camber for several special erection conditions, in order to be able to set the members first erected in their proper positions, to check the correctness of the erection work at various stages, and to effect any necessary adjustments. When making these figures, it is necessary to take into account the result of the distortion of the special erection members, and, when the span is cantilevered out from adjacent construction, of the distortion of the latter as well. The Williot diagram is generally employed in such cases, although the analytic method can be used when the position of but a single point is desired. The latter has the particular advantage that the movement of this point due to the adjustment of any particular member can be determined directly. If the shop lengths are computed by the approximate method, the positions which the various panel-points would occupy if the trusses were to rest on camber blocking should be figured, and the distortions from that outline due to the erection stresses should be calculated. If the exact method of determining the shop lengths is used, the outline of the truss when under the dead load plus one-half of the live and impact loads should be adopted as a basis, and the distortions due to the change in stresses from that condition to the erection condition should be figured. The exact method may be found to offer the more satisfactory solution, as the outline of the truss used as a basis therefor is generally rectangular, whereas the one employed in the other case is distorted into the circular shape. Either scheme, however, gives correct results.

THE CAMBERING OF CANTILEVER BRIDGES

The amount of live load for which a cantilever bridge should be cambered is hardly as important a matter as the like question is in the case of an ordinary simple truss bridge, because the dead load in any long span bridge (to which spans only this type of structure is suited) is much the larger portion of the total loading. The effects of secondary stresses can, in general, best be provided for by cambering for a live load of an amount lying between the total value and one-half thereof. When the floor line and the chords are more or less arched, the smaller value is entirely satisfactory; but when they are nominally straight, it will be best to camber for the total load, as the slight upward curve given to the horizontal lines thereby when the bridge is unloaded (which is the ordinary condition) will add to the appearance of the structure. In case changes of temperature can cause deflections, this fact must also be taken into account. In figuring the shop lengths of the various members, the dimensions of the structure under the load for which it is to be cambered are first determined. The shop lengths are then computed precisely as in the case of simple trusses, taking into account also the distortions due to temperature (if any). Care must be taken to see that the deformations of the anchorages are not overlooked. Next the outline of the truss under no load is figured. This is necessary for the assembling of the trusses in the shop, if this procedure is followed, and for the erection of the anchor arm, which is built on falsework. Then the outlines of the truss under various erection conditions are figured, in order that the correctness of the work may be tested at various stages, and any necessary adjustment made. This is of particular importance in case that the suspended span is cantilevered out from the cantilever arms.

THE CAMBERING OF STEEL ARCHES

As was previously mentioned, the only need for camber in an arch is for the purpose of preventing a nominally straight grade line from sagging. Hence, when the grade line is arched more or less, no camber is necessary, and the shop lengths of the various members can be made equal to their nominal lengths, allowing for the play in the pin-holes if the structure be pin-connected. If the grade line be nominally straight, the arch itself still requires no cambering, and the shop lengths thereof are to be determined as above; but the deflection of the arch under full load and maximum assumed fall of temperature should be figured, and the floor-line cambered sufficiently to make it straight under that condition. This will affect merely the members carrying the floor. For spandrel-braced arches, it will be necessary to lay out the top chord to follow the curve of the cambered floor-line rather than as a straight line. If the arch is erected on falsework, no camber diagrams will be required, unless it be a hingeless or two-hinged arch which it is desired to connect under certain special conditions; but if it be erected by cantilevering out from adjacent spans or from the banks, such diagrams must be drawn for various erection conditions, in order that the first members may be set properly, and so that the correctness of the work may be tested at intervals, and any necessary adjustments effected. In making these diagrams, the distortions of adjacent spans or of the anchorages or other special erection devices must be duly considered.

The Cambering of Suspension Bridges

In general, it will be best for a suspension bridge to have its nominal outline under dead load and shop temperature. The shop lengths of the cables and the members of the stiffening trusses are first computed, and then the outline of the stiffening truss under no load. Finally, the cambers of the cables and stiffening trusses under various erection conditions are figured, in order that the correctness of the erection work may be checked at any time and any necessary adjustments made.

The Cambering of Swing Spans

Theoretically, each arm of a swing span should be so cambered that any nominally straight chord thereof will be straight when the full live (plus impact) load is on that arm only. However, as in the case of a simple span, it is practically better to use a somewhat smaller camber. A satisfactory amount is obtained by assuming the nominally straight chords to be straight when full live (plus impact) load covers the entire span; and as the stresses for this condition of loading are always computed when the truss is designed, it will be a comparatively simple case of loading to figure for.

In finding the shop lengths of the various members, the entire truss (including both arms and the centre tower) is assumed to be of nominal outline under full dead load plus live plus impact loads, the uplift from the wedges being taken as the maximum amount which is considered to act when the top and bottom chords are at the same temperature. The shop lengths of all of the members are then calculated in precisely the same manner as was previously explained for simple spans. The camber under the no-load condition is then figured for use in assembling the trusses in the shop, if this be specified, and in erecting the bridge, if this be done on falsework. If the erection be accomplished in any special manner, additional camber diagrams for various erection conditions may be necessary, in order to check the correctness of the erection work at various stages and to make any needed adjustments. It will also be necessary to draw a camber diagram for the condition of span swinging, in order to be certain that sufficient movement has been provided for in the end wedges or toggles. In drawing the last-mentioned diagram, the effect of the top chord's having a higher temperature than the bottom chord is to be duly considered.

The Cambering of Vertical Lift Spans

The span itself should be cambered in precisely the same manner as a simple span, and for the same condition of loading. It will be necessary also to figure the lengths of the chords under dead load only, as that will be the loading at the time when the span is operated, and the one which will determine the location of the guides. The possible changes in the chord length due to temperature must be computed, in order to determine the play in the guides and the clearance with the towers when the span is operating; and the changes due to the live load are also needed in order to obtain the same information in respect to the guides and the centering castings when the span is seated. In this connection, it should be noted that under temperature changes both the top and the bottom chord points at the fixed end of the span remain stationary and those at the expansion end move; but when the live load comes on the bridge, the bottom chord point at the fixed end remains stationary and the one at the expansion end moves, while the top chord point at the expansion end remains practically stationary and the one at the fixed end moves.

The cambering required for the tower will depend on the type thereof and the manner in which it is supported. The several ordinary types will now be considered.

When the tower is of the four-leg type resting on masonry, with each column carrying the load of one sheave, no camber is necessary. If the rear legs rest on a span (which construction is very unlikely), the lengths of the tower legs will have to be such that the tower will be vertical when the supporting span carries dead load only. If the tower is to be erected after the supporting span has been swung and has received the larger portion of its dead load, it should be set vertical; but if it is to be erected while the said span is on falsework, or before it carries most of its dead load, it will have to be erected leaning forward by such an amount that when the full dead load comes on the span, the tower will assume a vertical position.

When the tower is of the single-bent type resting on masonry, no camber is necessary. If, however, the columns are riveted to the end of a plate girder or truss span, the connections must be so detailed that it will be vertical when the said span is under dead load. The remarks above given concerning the erection of towers of the four-leg type will apply to this style of tower as well.

In the usual type of tower a front column carries the entire load from the sheave, and is braced to a light rear column by a system of webbing. This rear column takes practically no load, and is generally supported by an adjacent fixed span, while the front column rests directly on a pier. The distance between the front and the rear columns of the tower at the top is quite small, say eight (8) or ten (10) feet, while that at the bottom is much greater. When, as is usually the case, the fixed span is a truss span, the rear leg is supported at the hip-point, if the tower be of ordinary height, and at the next top chord point, if the tower be very high.

It is evident that the tower must be so cambered that when the full load is on the sheave and the dead load only is on the supporting span the front column will be vertical. The proper shop lengths of the various members are determined in the following manner: The position of the panel-point which carries the rear leg is first found for dead load only on the span. Then the nominal outline of the tower is settled, using the position just figured for the panel-point at the bottom of the rear leg. The lengths of all members as given by this diagram may be called their nominal lengths. The theoretic shop lengths of all panels of the rear leg and of all bracing members will be equal to their nominal lengths,

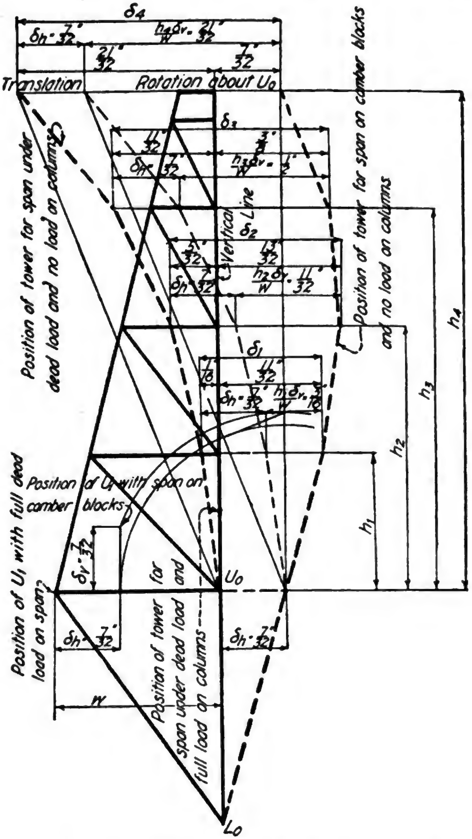

δ_4
$\delta_h = \frac{7}{32}''$
$\frac{h_4\delta_v}{W} = \frac{21}{32}''$
$\frac{21}{32}''$
$\frac{7}{32}''$
Translation
Rotation about U_0
δ_3
$\frac{11}{32}''$
$\frac{3}{8}''$
$\delta_h = \frac{7}{32}''$
$\frac{h_3\delta_v}{W} = \frac{1}{2}''$
Vertical Line
δ_2
$\frac{5}{32}''$
$\frac{13}{32}''$
$\delta_h = \frac{7}{32}''$
$\frac{h_2\delta_v}{W} = \frac{11}{32}''$
δ_1
$\frac{1}{16}''$
$\frac{11}{32}''$
$\delta_h = \frac{7}{32}''$
$\frac{h_1\delta_v}{W} = \frac{5}{16}''$
Position of tower for span under dead load and no load on columns
Position of tower for span on camber blocks and no load on columns
Position of U_1 with span on camber blocks
Position of U_1 with full dead load on span
$\delta_v = \frac{7}{32}''$
Position of tower for span under dead load and full load on columns
U_0
L_0
W
$\delta_h = \frac{7}{32}''$
$\delta_h = \frac{7}{32}''$
h_1
h_2
h_3
h_4

FIG. 33*a*. Camber Diagram for Tower of Vertical Lift Bridge.

while the theoretic shop lengths of the various panels of the front column are determined by adding to their nominal lengths their deformations under the load from the sheave. The actual shop lengths are found by expressing the theoretic shop lengths to the nearest thirty-second ($^1/_{32}$) of an inch.

The camber of the tower is next to be figured. Its nominal outline is already known; and from that its position when the supporting span is under dead load only and no load is on the sheave is determined by a Williot diagram, using for this purpose the differences between the nominal lengths and the theoretic shop lengths, as the connections will all be riveted. If the tower is not to be erected until after the supporting span has been swung and has received practically its full dead load, this diagram is all that is necessary. In case the tower is to be erected while the span is still on falsework, or before it has received its full dead load, an additional camber diagram is to be constructed as follows: The movement of the panel point at the bottom of the rear leg from its position when full dead load is on the supporting span is first figured, this movement being upward and toward the lift span. Let the horizontal and the vertical components of its motion be called δ_h and δ_v, respectively. The resulting motion of the tower may then be thought of as consisting of a forward movement of the entire tower (except the foot of the front column) equal to δ_h, and a rotation, about a point in the front column at the elevation of the bottom of the rear leg, through an angle $\frac{\delta_v}{w}$, w being the distance from the centre of the front column to the centre of the rear column at this point. The total forward movement δ of any point distant h above the bottom of the rear leg will, therefore, be given by the formula,

$$\delta = \delta_h + \frac{h\,\delta_v}{w}. \qquad \text{[Eq. 13]}$$

These movements, δ, of the various points are, of course, from their positions in the camber diagram previously drawn, not from their positions in the nominal outline.

Fig. 33*a* illustrates the application of the above methods to the drawing of the camber diagram of a tower for the supporting span under full dead load, and also on the camber blocks.

CHAPTER XXXIV

PROTECTION OF METALWORK

ONE of the serious problems confronting the bridge engineer is that of protecting metalwork under all conditions and in all places. Many observations, tests, and investigations have been made in the endeavor to find an effective means of protecting steel structures. A record of such a series of tests and their results is to be found in Vol. XIV of the *Proceedings* of the American Society for Testing Materials. An excellent *résumé* of the latest developments and conclusions with a brief synopsis of results accomplished by that society and other investigators is given in Vols. 15 and 16 of the *Proceedings* of the American Railway Engineering Association.

That the rusting of steel is due to an electrolytic action is becoming more thoroughly established with advancing research work. The lack of homogeneity in the metal furnishes a difference in potential between two contiguous portions, and the presence of water or moist air starts the said electrolytic action. Every particle of surface impurity, whether consisting of carbon, cinder, or oxide, forms with the iron in the presence of water a galvanic couple or minute battery. It is impossible in the present state of the art to eliminate these impurities; hence, for a practical remedy, we must look for a protective coating that will effectively shut out all water and air. The most common means of providing a coating is that of painting the structure.

Some experiments indicate that different ingredients of paint have different effects upon the metal. Some seem to act to as "inhibitors," some as "neutrals," and some as "stimulators" of rust. This is to be expected, for the accumulated experience of mankind shows that different materials have different properties and that, because of this variation, some of them must approach any given standard closer than others. It has also been found that these ingredients vary in their ability to exclude and to shed moisture. A material may exclude it for the time being, but the surface character thereof may be such that the dampness will cling to it until evaporated or absorbed; or the surface may be such as to make the moisture run off quickly. A "shedding" pigment may be a greater absorber of moisture than it is an "excluder" and yet in some situations prove the better protection.

Different pigments have different coefficients of expansion and drying and different moduli of elasticity. In some cases the finishing coat will crack along the priming coat; and the liability of some of the best "inhibitors" to crack is so great as to warrant their rejection. It has been found that the chemical composition of the pigment may be affected by light, or heat, or moisture, or gases, so that it would fail under certain conditions, while under more favorable ones it would give good service.

The vehicle is as important as the pigment. Some vehicles which have an objectionable degree of porosity may, with the addition of a pigment filling the voids, produce a successful protective coating.

Investigations indicate that bituminous coatings protect metal better for a short time than any other kind; but that the action of the sunlight quickly destroys their life and renders them valueless as a protective material.

The foregoing facts lead to the following tentative general conclusions:

1. Priming coats should always be inhibitors regardless of their qualities as excluders or shedders.

2. Second coats should, preferably, be excluders; and the nearer they approach to the inhibitor class the better.

3. The third or finishing coat should be a good shedder, without regard to its properties as an inhibitor, although the closer it approaches that class the more satisfactory it will be.

4. Due consideration must be given to the conditions and the deteriorating influences to which the structure will be subjected.

5. The value of a pigment is affected by the process of manufacture, so it may appear as an inhibitor, or a stimulator, or any of the various gradations between; hence it is necessary to determine its process of manufacture before using it as a priming coat.

Some earlier experiments indicate that carbon and graphite pigments and linseed oil are stimulators and, hence, are not desirable for the priming coat; while zinc and zinc-lead pigments are good primers. Later experiments are somewhat contradictory. It is also shown that pigment of the lead basis may belong to either class, depending on the process of manufacture.

The most commonly used paint vehicle is linseed oil. This alone does not make a good priming coat, as it is a stimulator and also a poor excluder. Attempts have been made to find another oil that would better meet the conditions. An account of such a test on 59 different vehicles is given in the *Engineering News*, Vol. 73, page. 70 In this test each of the different vehicles was mixed successively with four different pigments, viz., red lead, iron oxide, graphite, and carbon. The result of the experiments showed that linseed oil with the four pigments in turn averaged highest for one coat, one year; next to the highest for one coat, two years; next to the highest for two coats, one year; and next to the highest for two coats in two years. In no case was there very much difference between the linseed oil paints and the paints mixed with bleached varnish oil or with extra-bleached winter fish oil. Those mixed with the two last mentioned oils gave slightly higher average results. For the present, at least, it seems that we must accept linseed oil as the best available vehicle and endeavor to make up its deficiencies by the judicious selection of pigments for the several different coats. Hence for priming coats, a neutral or inhibiting pigment such as red lead or a zinc oxide should be adopted. So far, red lead, as a primary coat, has proved about as effective as any other pigment in the retardation of rusting.

Several years ago the author sent a letter of inquiry to the Chief Engineers of twenty of the principal railroads of the United States requesting a statement as to the kinds of paint they used and the lengths of life thereof. A canvass of the replies showed that fourteen roads used red lead; five employed carbon only, and three had adopted carbon in addition to red lead. One road employed iron oxide exclusively, and one used it in addition to red lead; nine employed graphite (one exclusively, one in connection with carbon, and seven in connection with red lead); two roads used lamp black in connection with red lead, and one road employed asphaltum in addition to red lead. As these roads had done, on their own account, some experimenting and investigating before adopting a particular paint or combination of paints, it is to be inferred that the process of elimination left each road with the paint best adapted to meet the conditions prevailing on that system. Hence the fact that different paints were adopted by the several roads leads to the conclusion that no one paint will suit all conditions. However, the concensus of opinion is largely in favor of red lead for the priming coat. It should be finely ground and mixed in the proportion of 25 to 30 lbs. of lead per American gallon of linseed oil; and the mixing should be done just before the time of application. The pigment should not be allowed to settle. As it is somewhat difficult of application, a good workman with a stiff brush is required to put it on.

For the second and third coats, which are to be applied after the metal has been erected, carbons, lamp blacks, or graphites should be used, as they are good excluders and shedders of moisture. Each coat should be of a distinctively different color from the others, so that it can readily be ascertained whether the entire area has been covered with one coat before a subsequent one is applied. It will facilitate the inspection work to have a small pocket mirror in order to reflect light in the places difficult of access. Surfaces in contact should be given their two field coats before being assembled in place. It is important that no painting be done in wet or freezing weather.

In applying paint to new work, it is essential that the surface be cleaned from mill scale, mud, rust, or grease, that the metal be dry, and that the temperature of the surrounding air be above the freezing point of water. The more common method of cleaning is that of using steel scrapers and wire brushes. This is a slow process if thoroughly done. An acid bath is sometimes employed, followed by another of milk of lime, and then by a thorough washing with hot water. This method is expensive; and it is also unsatisfactory, unless great care is taken to remove all acid. It is also apt to leave the metal wet and thus allow rusting to commence before the paint is applied. Another method which has many advantages is that of the sand-blast. This removes rust, dirt, and mill scale, and leaves the metal clean, dry, and free from acid, so that painting can follow immediately. It gives a superior surface to that produced by the wire-brush method, and involves much less time and cost where there is a large amount of cleaning to be done. On account of the thoroughness of the cleaning which it effects, the sand-blast secures a stronger adhesion of the paint to the metal, especially upon surfaces that have been pitted by rust. Difficult corners and re-entrant angles which are inaccessible by hand can readily be cleaned by this method. It has been found that an air-pressure of eighty pounds per square inch gives the most economical results. About three square feet per minute can be cleaned by adopting this intensity. As the pressure ought to be constant, a small cylindrical receiver about 18 inches in diameter by 36 inches in length should be installed in the air main in order to steady the intensity and to catch the condensation.

The cleaning should be followed promptly with a coat of paint; for rusting is apt to set in quickly. Mill scale can be removed more easily with a sand-blast, if it has been allowed to rust somewhat.

The cleaning of old structures preparatory to painting is usually done with scrapers and wire brushes. Unfortunately, the need for thorough cleaning and its relation to the life of the paint that is applied subsequently are not sufficiently appreciated. If they were, the use of a portable sand-blast outfit would almost invariably be substituted for the inefficient method of scrapers and brushes. Before repainting a structure all loose paint, rust, and dirt should be thoroughly removed; but any old paint which adheres firmly to the steel may be allowed to remain. The parts cleaned should have a rust-retarding coating promptly applied (the same as that for the shop coat); then, when this is thoroughly dry, the entire surface should receive the next coat.

It will be found that usually the horizontal surfaces of a structure show the first signs of paint deterioration. It would be economy to replace the paint on these portions oftener than on the vertical surfaces. As the cost of proper cleaning is greater than that of the painting, it is better to do less cleaning and more frequent painting. Too much stress, however, cannot be laid on the matter of thorough cleaning before applying the paint.

Advantage may be taken of the fact that light colors absorb less radiant energy from the rays of the sun than do dark colors, and hence reduce the range of temperature variation in the metal. For long swing spans, especially in hot countries, a light colored paint would prevent to some extent the heating of the top chord and the consequent dropping of the ends of the span.

There is another method of metal protection which has been growing in favor of late years, and that is the encasing of the metal in concrete. The parts to be protected are covered first with a wire mesh and a temporary wooden form is built around the same, after which a rich concrete is poured in so that the member is completely covered. Another process is to cover the member with the wire netting, and, by means of an air gun, squirt a rich mortar against the metal. This coating can thus be made several inches thick, if so desired. Encasement of this type has been placed at a cost of about eight cents per square foot per inch of thickness. Results of experiments along this line will be found conveniently recorded in Vol. 15, page 426, of the *Proceedings* of the American Railway Engineering Association. A dense coating is desirable in order to prevent absorption of water and possible beginning of electrolytic corrosion.

In some cases where a concrete casement has been exposed to seepage from the floor of a bridge, it has cracked and allowed the wire mesh reinforcement to rust through, with the result that the concrete has eventually dropped off and left the steel exposed. This trouble can be avoided by properly waterproofing the deck. It has also been found that where a concrete encasement is close to the exhaust of the locomotive, the blast wears the covering away rapidly so that additional protection is needed. This is provided by interposing between the blast and the concrete a steel plate, which will last several years and then can readily be replaced by another.

While much valuable information on the subject has been accumulated of late years, there is still need for more investigation and experimentation along the line of effectively protecting steel structures before the last word can be written thereon.

The author is often asked the question as to what special brands of paint his experience would lead him to consider the best for metalwork; and he always answers to the best of his knowledge and belief, notwithstanding any risk he may run of being charged with favoritism—or worse. Most engineers are somewhat chary about expressing an opinion concerning the relative values of the materials that are used in competition on their constructions; but the author is of the belief that an interchange of technical knowledge and experience of all kinds is of the utmost importance to the general welfare of the engineering profession. On this account he feels that he would not be justified in completing this chapter without presenting a statement of his personal experience with the various kinds of bridge paints; and he herewith does so without offering any apology for such action.

The question as to which is the best of all the standard bridge paints is a difficult one to answer, for some otherwise excellent paints will fail under certain peculiar conditions. For instance, during quite a long period, over a decade ago, the author adopted on many of his structures the Detroit Superior Graphite paint, as it gave quite satisfactory results, having a durability of about five years; but in his bridge over the Red River at Alexandria, La., and in a number of spans on the Vera Cruz and Pacific Railway of Mexico it failed utterly in two or three years, probably because of the warm, moist, climatic conditions. On this account he ceased using it; but lately he has been informed by an agent of the manufacturers that his experience and consequent action constituted the reason for their making some extensive and elaborate experiments upon how to manufacture their product in different ways so as to suit climates of all kinds, and that they have succeeded in solving the problem to their own satisfaction. They now employ a special formula for each type of climate, and are ready to guarantee the durability of their output when employed in the proper country.

When building a bridge at Boca del Rio near Vera Cruz, Mexico, within a very short distance from the Gulf and quite close to the water's surface, the author recognized the necessity for the adoption of the best possible preservative against the ravages of salt water in the tropics; consequently he proceeded to make an investigation of what knowledge of the subject had been accumulated by other engineers. He learned that the late A. J. Tullock of Leavenworth, Kansas, before building the large wharf at Tampico, Mexico, had experimented in a very thorough and practical manner upon twenty of the best known brands of bridge paints, and had found that one of them was far superior to all the others. It was called Z. P. Leiter's Air-Drying, Salt-Water-Proof Paint, and was manufactured in Chicago. Upon the strength of Mr. Tullock's recommendation the author applied it to the bridge mentioned; and five years later he was informed that it was in as good condition as it was the day it was put on Having occasion afterward to build some important bridges for the City of Vancouver, British Columbia, over False Creek, an inlet from the ocean, he specified the Leiter paint for both shop and field coats. The metalwork was manufactured in winter at the shops of the Dominion Bridge Company near Montreal; and the coldness of the steel prevented the paint from adhering to it, consequently its employment had to be abandoned. It has been reported that this paint is no longer procurable, as its manufacturer is dead and its formula is lost. If such be the case, the fact is certainly to be deplored; because the paint undoubtedly was a most excellent covering for metalwork exposed to the salt water.

On the Mexican bridges, before mentioned, five different kinds of paint were tried so as to find the one best suited to the climate of the *tierra caliente*—the two already named, a red lead paint mixed with Leucol oil, and two others the names of which have been lost. All but the Leiter paint failed very quickly.

Johnson's Magnetic Iron Oxide Paint gave the author good service on several important bridges many years ago, but the manufacturer thereof once adulterated his product to meet some close competition for a large piece of elevated railroad work constructed by a friend of the author's, hence the latter has never since used it, in spite of considerable pressure brought to bear on him and of many oft-repeated promises on the part of the manufacturer "to be good" in the future.

Certain special bridge paints placed upon the market by Lowe Bros., Sherwin-Williams, Toch Bros., and the National Lead Company give excellent results; but as most paint companies manufacture their products in varying degrees of excellence in order to suit the purses of all purchasers, the author cannot personally recommend any of their brands, as he has not used them to any great extent. There are, however, two bridge paints that have always given satisfactory results on the author's constructions, viz., the Goheen Carbonizing Coating and Nobrac; but he has never used either in the tropics. These two paints seem to be always uniform in quality and very dependable. The manufacturer of the former makes a practice of guaranteeing his product for ten years, provided he be allowed to place one of his own trained men on the work to supervise its application. Such a guarantee, at first thought, appears to be an excellent idea; but when it is adopted there generally arises so much friction between the paint inspector and the contractor for erection as to make the life of the resident engineer a burden to him.

Red lead paint manufactured by the modern process of grinding in linseed oil is certainly the best of all paints for the shop coat, but it must be honestly compounded, honestly mixed, and honestly applied in order to be truly effective. For the field coats of bridge metal there is not much choice between several of the best known brands of carbon or graphite paints, including especially those above mentioned.

CHAPTER XXXV

WOODEN BRIDGES AND TRESTLES

ALTHOUGH wooden bridges in some form or other have been used for many centuries, and although their design has finally attained to a semi-scientific stage of development, they are now rapidly becoming a thing of the past. The advent of cheap steel, the introduction of reinforced concrete, the great increase of live loads, and the diminishing supply and deteriorating quality of timber have contributed to this result. However, in some special situations where timber is abundant, of good quality, and reasonable in price, or where a structure is wanted for a temporary purpose, it is still true economy to build wooden bridges. Broadly speaking, such structures may be divided into two classes, viz., truss spans and trestles.

Truss spans of timber construction are used for openings that are too large to admit of the load being carried by any available timber beams. They may be of either the deck or the through type. The chords are composed of timbers placed side by side with spacers between and bolted together so as to act somewhat as a unit. The spaces afford drainage and ventilation and thus retard the growth of the fungi that cause decay. The web members are sometimes made of planks forming a lattice work of the multiple cancellation type like the Town truss, as shown in Fig. 1*f*; and in other cases they consist of two systems of diagonals arranged to take compression and crossing at the central point of panel, with steel rods for verticals, similar to the Howe truss shown in Fig. 22*p*.

Lateral bracing of timber of the same type of cancellation located in the horizontal planes of the chords is employed. Effective knee bracing and portal bracing are difficult to secure. Care must be taken to see that the timbers of the bottom chords are properly spliced for tension at joints with steel plates of sufficient net section to develop the net tensile strength of the wood; also that cast-iron bearing or angle blocks are employed to distribute the stress from struts over sufficient bearing area of the contiguous timber to prevent the fibres thereof from being crushed. While timber can develop considerable resistance to forces applied in the direction of the fibres, it has relatively little strength to resist tension across the grain, compression across the grain, or longitudinal shear. Its resistance is greatly affected by moisture. During wet weather the wood becomes soft and squeezes together, if the transverse load on it is excessive. When it dries out it shrinks, while the load coming on it holds it down to its smallest dimension; and then when wet weather comes again, it swells out at right angles to the direction of the load. These alternate shrinkings and swellings soon destroy the fibre and lessen materially the strength of the timber. It is essential, therefore, to adopt a small unit of compression across the grain. To obtain these low unit pressures it is necessary to use iron bearing plates of ample proportions. Correct detailing of splices and joints will do much to prolong the life of wooden structures.

The floor system is supported on cross beams every few feet resting on or suspended from the lower chords in case of a through span, or lying upon the upper chords in case of a deck span. This loading produces a bending in the chords that must be taken into consideration and combined with the direct stress when proportioning the section of the member.

Timber trestles can be employed where the conditions admit of using supports about 14 feet apart to carry the timber beams. These trestles may be divided into two general classes, viz.:

First, Pile-trestles, or those in which each bent is formed of several piles, a cap, and transverse sway-bracing; and,

Second, Framed trestles, or those in which each bent is composed of squared timbers framed together and braced.

Owing to the excessive length of piles required for greater heights, pile-trestles should rarely, if ever, exceed thirty feet in height; while framed trestles, if properly designed for rigidity as well as for strength, may be carried up to much greater heights, the economic limit being probably about one hundred feet.

PILE-TRESTLES

THE bents of a pile-trestle should contain at least four piles each. Where the trestle does not exceed ten feet in height, the piles may be driven vertically, and no sway-bracing need be used, provided that the piles have a good penetration in reliable material. For greater heights of trestle than ten feet, the two outer piles of each bent should be given a batter of from two to three inches to the vertical foot. Each bent should also be braced with one or two sets of sway-bracing, each composed of two 3″ x 10″ yellow-pine diagonals, thoroughly bolted to the piles, wherever they cross them, by $\frac{3}{4}$″ bolts. Wherever the piles are of irregular sizes, they should be trimmed off so as to make the diagonal bracing fit properly.

The piles for such bents should be so spaced laterally as to give great transverse rigidity to the structure, and at the same time afford ample support for the caps. A good spacing is as follows: Distance from centre to centre of outer piles, 11′ 0″; distance from centre to centre of two inner piles, 4′ 6″.

The caps should be at least 12″ x 14″ x 14′, placed on edge and attached to the piles by means of $\frac{7}{8}$″ drift-bolts.

For ordinary pile-trestles in fairly firm soil no longitudinal sway-bracing will be required for heights below ten feet; but for heights between ten and twenty-two feet, one-story, longitudinal sway-bracing should be used in every fifth panel, so as to prevent the structure from moving longitudinally as a whole because of thrust of trains. For heights greater than twenty-two feet, each alternate panel should be braced longitudinally by two-story bracing, so as to hold the piles at mid-height, and thus strengthen them as columns; and the transverse sway-bracing for these cases should also be two-story for the same reason.

For ordinary pile-trestles up to twenty-two feet in height the panels should be a trifle less than fourteen feet in length, while for greater heights either the same length may be used or alternate panels may be made from twenty-four to twenty-eight feet long by trussing the stringers, according to which of the two methods is the more economical.

The stringers under each rail should be built of three runs of timber, generally sixteen inches deep, the sizes being determined from the loading by using an intensity of two thousand pounds (or less) for the extreme fibre, when impact is included. The stringer timbers are to be separated from each other at the panel points by means of timber packing-blocks, which are to serve also as splice-timbers. These timber blocks should be at least three inches thick and six feet in length, and should have at least four bolts through them. They are to be separated from the stringers by small cast-iron fillers three-quarters of an inch thick, so as to prevent the timbers from coming in direct contact with each other. The splice-timbers must be made wide enough to project an inch or two below the bottoms of stringers, and must be notched over the caps so as to hold the stringers firmly in place. The distance from centre to centre of middle stringers should be five feet. Intermediate cast-iron separators with bolts should be used between adjacent stringer-timbers, at distances not to exceed five feet centres. The length of the stringer timbers for ordinary trestles should be twenty-eight feet, so as to extend over two panels, and thus stiffen the floor system materially. The ties should be 8″ x 8″ x 10′. They should be dapped over the stringers at least one-half inch and spaced thirteen inches from centre to centre.

Inside and outside guard-rails should be used for all trestles, and at each end of every trestle some satisfactory kind of re-railing device should be employed. The outer guard-rail should be made of a 6″ x 8″ timber laid flat and dapped one inch on the ties. The inner faces of the outer guard-rails should be spaced not less than twelve inches from the gauge-planes of rails. The inner guard-rail should be 5″ x 8″ laid flat, and dapped one inch on the ties. The outer faces of the inner guard-timbers should be placed five or six inches inside the gauge-planes of rails. Both inner and outer guard-rails should be bolted to alternate ties by three-quarter-inch bolts, which must pass through the stringers also.

The heads of these bolts should be countersunk into the tops of the guard-rails by means of cup-shaped washers.

Framed Trestles

For trestles of greater height than thirty feet, and for less heights under certain conditions, it will be necessary to use framed bents. The foundations for these may be provided by driving piles and cutting them off above the ground, by using timber sills, or by building small masonry piers.

In any such trestle it will be necessary to brace the structure thoroughly, both transversely and longitudinally. All framing of bents should be done in such a manner as to tie all parts firmly together.

For very high trestles it will be economical to increase the lengths of alternate panels to twenty-five or even thirty feet, and truss the stringers.

The longitudinal bracing should consist of diagonals of timber of suitable dimensions, in alternate panels, with horizontal struts made continuous throughout all panels. In addition to the transverse and longitudinal bracing previously described, all trestles on sharp curves should be provided with a lateral system composed of timber diagonals spiked to caps and to bottoms of stringers.

In designing wooden bridges it would be well thoroughly to consider the question of renewals, and where possible to provide facilities for making them. To accomplish this, sufficient strength should be provided so that any one piece of timber can be taken out and replaced without endangering the structure. In this way, if necessary, the bridge can be made serviceable for many years. Some railroad companies put a solid timber floor on their trestles and cover it with ballast to support the track. This affords a fair protection against fire, decreases the noise, and provides a better track than does the common type of wooden floor. On the other hand, it is rather expensive to build and to maintain, and is somewhat difficult to inspect. The stringers are placed close together, thus forming a solid floor; or else they are covered with 3″ creosoted planks.

To prolong the life of a wooden structure creosoted piles and creosoted timbers may be used. When it is necessary to saw off the ends of the piles or any of the timbers, the cut surfaces should be painted thoroughly with creosote. Specifications covering the treating of timber for preservation will be found in Chapter LXXIX. However, it is becoming more and more inadvisable to build wooden trestles except of the most temporary character, as the live loads are getting to be so great that the materials in such structures are liable to crush under excessive concentrations. Wooden trestle construction may be employed to advantage on new work to take the place of a high fill until the railway is completed when work trains can be run over it so as to dump earth into and around the structure and thus build the new embankment at moderate expense.

The kinds of timber best adapted for wooden bridges are long-leaf yellow pine, Douglas fir, cedar, and Western hemlock. Those for piling are the ones just mentioned and, in addition, white oak, burr oak, tamarack, and cypress.

The general specifications for designing given in Chapter LXXVIII will apply to wooden bridges; but will need to be supplemented by the following intensities of working stresses for timber, when impact is included:

For the higher grade woods, such as long-leaf yellow pine, Douglas fir, Pacific Coast cedar, Western hemlock, and white oak:

Tension	2,000 lbs. per square inch
Bending on extreme fibre	2,000 lbs. per square inch
Shear with the grain	280 lbs. per square inch
Longitudinal shear in beams	180 lbs. per square inch
Shear across the grain	1,600 lbs. per square inch
Compression with the grain	2,000 lbs. per square inch
Compression across the grain:	
For white oak	750 lbs. per square inch
For the other timbers	400 lbs. per square inch
Compression on columns:	
Under 15 diameters	1,500 lbs. per square inch
Over 15 diameters	$2{,}000 - 35\frac{l}{d}$ lbs. per sq. in.,

where l = length in inches, and d = least dimension of section in inches.

For the lower-grade woods, such as the soft pines, spruce, tamarack, and redwood:

Tension	1,500 lbs. per square inch
Bending on extreme fibre	1,500 lbs. per square inch
Shear with the grain	160 lbs. per square inch
Longitudinal shear in beams	120 lbs. per square inch
Shear across the grain	1,200 lbs. per square inch
Compression with the grain	1,500 lbs. per square inch
Compression across the grain	250 lbs. per square inch
Compression on columns:	
Under 15 diameters	1,100 lbs. per square inch
Over 15 diameters	$1{,}500 - 25\frac{l}{d}$ lbs. per sq. in.,

where l and d have the same values as before.

In grouping Douglas fir with long-leaf yellow pine and the other higher grades of woods, the author recognizes that he is laying himself open to possible criticism, for some of the authorities rank it much lower than the long-leaf yellow pine, while others, such as the American Railway Engineering Association, specify but little difference between the two woods. From an extensive experience in British Columbia, Washington, and Oregon, with both Douglas fir and Pacific Coast cedar, the author is inclined to consider these timbers about as good and reliable as any that can be obtained for the building of wooden bridges and trestles. Their quality is very uniform and they can be purchased of large size and great length.

Allowable stresses for nails, screws, dowels, and drift pins are given on pages 660 and 661 of Merriman's "American Civil Engineers' Pocket-Book" (Second Edition).

For the detailing of wooden bridges and trestles the reader is referred to Jacoby's excellent book entitled "Structural Details or Elements of Design in Heavy Framing," and to Foster's well known standard "Treatise on Wooden Trestle Bridges."

In Figs. 53*c* and 53*d* are given the costs of wooden trestles per lineal foot of structure for various prices of timber per M. ft. B. M. in place. These figures are sufficient for Class 50 live load, but for heavier loads they must be increased ten or fifteen per cent so as to allow for a closer spacing of the bents and for a possible adoption of heavier posts or piles, or both. For Classes 55 and 60 it would suffice to add ten per cent and for Classes 65 and 70 fifteen per cent. If piles of greater penetration than those indicated on the diagrams are needed, the value of the extra lengths thereof will have to be figured per lineal foot of trestle and added to the costs given on the curves. These costs were computed for structures on tangent or on curves of less than four degrees. For trestles on sharper curves it will be necessary to add from one to two dollars per lineal foot in order to provide for additional horizontal bracing.

Since the preceding was written, the following article has appeared in a New York newspaper:

"FIR LUMBER AND THE BRIDGE TRUST

"Although there is no relation between the two subjects so far as known, it is nevertheless a striking coincidence that at a time when some of the western states are entering protests against the so-called bridge trust, which has to do with steel work only, there is a renewal of interest in the same section, and especially in Oregon, in wooden bridge construction. In Iowa, it is alleged, the bridge trust favors and imposes old-fashioned methods while good roads and good bridges demand progressive ideas. In Minnesota the Legislature has taken cognizance of complaints against the method of granting contracts to the big bridge companies. It has been found that contracts for bridges at or about a certain price almost invariably go to one company, at a higher price to another, and so on, the presumption being that an agreement exists between the bridge construction concerns that is practically a pool. Since all the western states are constructing new, or reconstructing old highways in these days, the matter of bridges is a very important one. It is important not only as regards the first cost of the bridges but also as concerns their maintenance.

"Many engineers and builders have long insisted that wooden construction under proper conditions will outwear steel work. A late issue of the *Timberman* contains a testimonial to the lasting qualities of fir lumber in bridge construction from L. N. Roney of Eugene, Ore., who has superintended the building of bridges in Lane county of that state during an extended period. What he says of fir is, we think, of general interest. Speaking particularly of a wooden, Smith truss bridge, with a span of 230 feet across the Willamette River at Eugene, he says that when taken down two years ago, after it had served heavy traffic for thirty-seven years, its timbers were found to be absolutely free from all signs of decay. In his opinion, the bridge would have been good for as many more years had it been possible to renew the bottom chord and to replace the piers. The trouble was with the substructure, not with the bridge. Mr. Roney calls attention to two other bridges in the same county, of 250 feet span length, one built more than forty years ago, the other four years later, both of which are in splendid condition so far as the span timbers are concerned. They also are supported by wooden piers which must be replaced every ten years. From an experience which embraces the construction of more than forty wooden bridges, Mr. Roney draws the conclusion that a superstructure of Oregon fir, if built on permanent piers of stone or concrete, with the spans carefully protected from the weather, 'will outlast steel bridges, and certainly the upkeep in flooring, etc., is nothing in comparison to that of an uncovered structure.'

"It would be well for county commissioners and state highway commissioners to look a little more closely into the relative merit and cost of steel and wooden bridges. Good roadway improvement is often postponed because of the great expense of bridge construction. The question is, whether steel bridgework is preferable to wooden, or whether wooden construction, using home material and employing home labor, thereby avoiding contact with unscrupulous trusts and equally unscrupulous contractors, is not more economical on every ground."

From the preceding quotation it would appear that, on the Pacific Coast at least, there is still a chance for the continued building of wooden truss bridges. If they be properly constructed, thoroughly housed, and effectively maintained by truly water-proof roofing and siding, they would certainly be more economic than steel bridges of the same capacity, were it not for their one great characteristic weakness—liability to destruction by fire.

CHAPTER XXXVI

DRAW BRIDGE PROTECTION

Where there is much traffic on a river that is crossed by a bridge having a swing span, it becomes necessary to build a draw protection so as to prevent vessels or rafts from injuring either the draw span or its pivot pier. The rest piers at the ends of draw spans often require protection also. As draw protections are expensive and generally short-lived, it is well to omit them when this can be done with comparative safety; but when one does so, he lays himself open to extortion by unscrupulous persons who will intentionally wreck a worthless old vessel by running it into one of the piers or the swing span and sinking the craft, and then claim excessively heavy damages. This occurred once in the case of one of the author's Missouri River bridges; but the blackmailing scheme failed to work, for one of the interested parties gave such a mass of false evidence in his testimony that he lost his case.

A complete draw protection costs ordinarily from five thousand (5,000) to twenty-five thousand (25,000) dollars, according to the depth of water, velocity of current, character of river bed, length and width of span, variation of elevation between high and low water, unit values of the materials used, character of river traffic to protect against, frequency of passage of vessels and rafts, and comparative permanency of the construction. Except where the water traffic is very great, the tendency of the designer is to cheapen the construction as much as he dares; and this is generally advisable, yet one must draw the economic line beyond the place where the life of the protection is liable to be shortened for want of sufficient resisting capacity. A draw protection to be truly effective should be about two (2) feet wider on each side than the swing span itself in order to prevent the upper works of high crafts from striking the superstructure when the span is open. It is advisable to give the protection ample strength to resist all shocks without receiving any aid from the pivot pier, for it is generally bad policy to run the risk of injury to the latter. This suggestion is often violated by designers, who, in order to stiffen their otherwise weak protections, let them abut against the masonry of the pivot pier.

The correct theory in the designing of any draw protection is to give it great resilience instead of great rigidity so that, when struck a heavy blow, it will spring and not break. Because of this fact it is best always to build draw protections of timber. The usual and most satisfactory type is that composed of long piles, substantially capped, sheathed with thick planks, and effectively braced together by timbers in both horizontal and vertical planes. An example of this style of construction is shown in Fig. 36*a*, which illustrates the draw protection for a little highway and street railway bridge of the author's over a portion of the Fraser River between the City of New Westminster and Lulu Island, in British Columbia. In this case the water is not very deep, but the sandy bed of the river is quite liable to scour; and the water traffic, though not frequent, is heavy and destructive, for great booms of large logs pass through on their way to the sawmills. The bridge being located not far from the salt-water, the current runs both ways, hence the necessity for pointing both ends of the fender. In this type of construction it is generally necessary to protect all angles in the faces by firmly attached steel plates—especially is this advisable where the ice-run is great and destructive.

In locations where the *teredo navalis* and other sea worms operate, it becomes necessary to use piles that are impervious to their attacks. Some very hard woods from Australia and from the American tropics resist the worms effectively; but often such timber is not procurable, hence it is obligatory to use soft-wood piles and to creosote them as heavily as possible. They will then last for many years; but in time, as the creosote oil is slowly washed out, the teredo begins to get in its work, and then the life of the timber is limited to a few years or even to a few months.

Another type of draw protection consists of a timber crib at each end of the construction, filled with large stones, to support the outer ends of two wooden, Howe-truss spans that are built wide enough to protect the swing span. The inner ends generally rest on the caisson of the pivot-pier, or are attached to the pier itself, which, as before stated, is an objectionable feature of design. However, if the pier be large in diameter and well founded, it will be able to resist any shock that comes to it through the resilient timber truss spans. For the timber cribs filled with stones may be substituted pile noses similar to the ends of the draw protection already described; and near the pivot pier there may be placed pile bents to support the inner ends of the Howe-truss-span protection.

The groups of closely driven piles near the ends of the rest piers of the Lulu Island Bridge, shown in Fig. 36*b*, form the best kind of protection for those portions of a swing-span bridge, because they are entirely detached from the structure, have great resilience, and fend off most effectively all blows from passing vessels and rafts—moreover, they are inexpensive. As shown on the drawing, each fender consists of a group of piles driven in a circle around a central pile about as close as they will go conveniently, with their tops sprung together radially till they are in contact and bound tightly in place by heavy chains.

As draw protections become old, their lives may be prolonged by replacing decayed and broken timbers or piles; and it is legitimate to do this, because a partial failure of the construction by a blow from a passing craft would not be likely to involve serious injury to either the swing span or its pivot pier, for, in all probability, most of the force of the blow would be exhausted in breaking the protection. However, it would not do to carry this economic idea to extremes.

In all draw-protection work only good, sound, strong materials should be employed, and all connections should be most thoroughly made. Drift-bolts, screw-bolts, and spikes should be used liberally, for their cost is a bagatelle compared with the good they do by strengthening the construction. Hard wood planks, if procurable at reasonable cost, are advisable for the facing, but generally it is necessary to use soft wood piles.

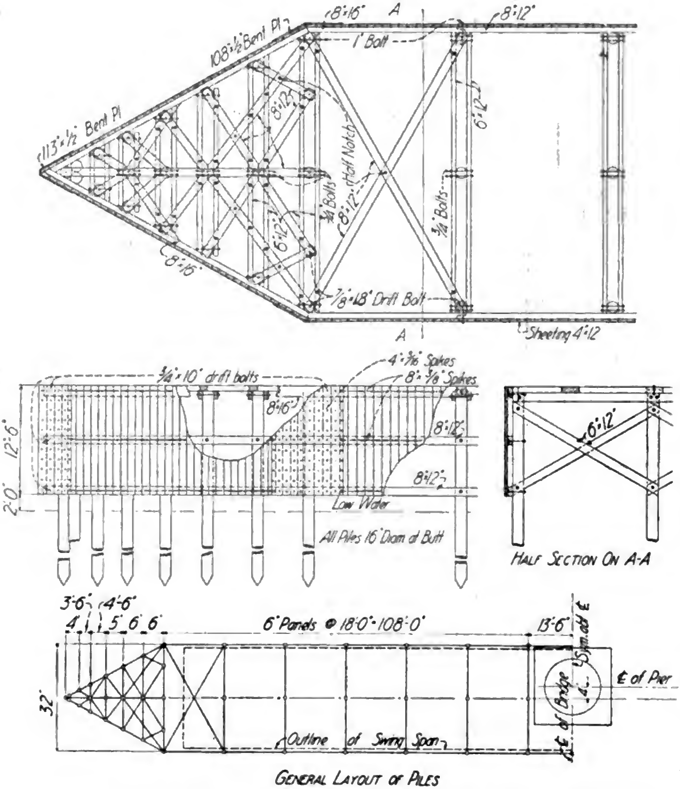
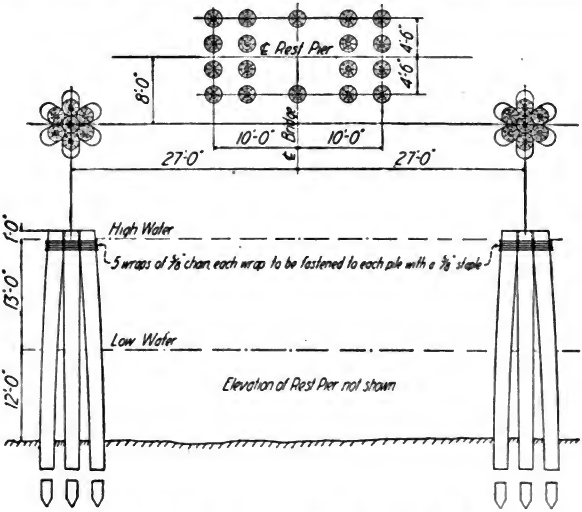


FIG. 36a. Draw Protection for the Lulu Island Bridge in British Columbia.

All piles should be unusually straight so as to permit the attaching of the waling pieces without shimming or undue cutting. In connecting the steel plates to the planking, the screw-heads should, preferably, be counter-sunk into the metal in order that there may be no projecting ends to be broken off.



℄ Rest Pier
8'-0"
4'-6"
4'-6"
10'-0"
10'-0"
℄ Bridge
27'-0"
27'-0"
1'-0"
13'-0"
12'-0"
High Water
5 wraps of 7/8" chain each wrap to be fastened to each pile with a 7/8" staple
Low Water
Elevation of Rest Pier not shown

FIG. 36*b*. Fender Piles for the Lulu Island Bridge in British Columbia.

For protecting the piers of the opening spans of vertical lift and bascule bridges the best and most economic method is to put a dolphin or group of cluster piles near each end of each supporting pier, and, in some cases, adding a single line of capped and faced piles along and close to the face of each of such piers.

CHAPTER XXXVII

REINFORCED CONCRETE BRIDGES *

ALTHOUGH the first patents for reinforced concrete were taken out some sixty years ago, as was stated in Chapter I, it has been only twenty-five years since it was first applied extensively to bridge construction. Its use has increased so rapidly of late, however, that it is today one of the most important materials which the bridge engineer has at his disposal.

The design of reinforced concrete construction during the earlier stages of its development was largely empirical, as the nature of the stresses was but imperfectly understood. In the late eighties more or less rational methods of analysis were evolved, but they were based on rather limited experimental data. As the use of the material increased, a great many additional tests were made; and by the close of the century most features of design were on a fairly firm basis, although there was still a wide difference of opinion on some important points. During the past fifteen years many experiments have been carried on by various investigators in an attempt to settle these uncertainties; and in the main they have been successful. No doubt the next decade will see some important additions to our knowledge of the subject, but it appears unlikely that there will be many radical changes in the methods of design now prevailing.

In 1903 and 1904 special committees were appointed by the American Society of Civil Engineers, the American Railway Engineering and Maintenance of Way Association (now the American Railway Engineering Association), the American Society for Testing Materials, and the Association of American Portland Cement Manufacturers, to investigate current practice, provide definite information concerning the properties of plain and reinforced concrete, and recommend factors and formulæ which should be used in designing. These special committees united to form a Joint Committee on Concrete and Reinforced Concrete. In 1909 that committee presented a progress report to the various societies which it represented. This report was incomplete and unsatisfactory in some respects, due largely to a lack of time for settling certain differences of opinions among the members of the committee. In 1913 a second progress report was made after all doubtful points had been cleared up and every-

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* All the original features of this chapter, except as otherwise credited, are the work of Mr. Shortridge Hardesty, as, in truth, is the whole chapter (of course, under the direct supervision of the author), excepting only that the data for field work were furnished by Mr. N. Everett Waddell, and that the entire chapter after completion was given a thorough, systematic, and detailed check by Mr. Herman H. Fox.

thing had been put into satisfactory shape. This 1913 report is to be found in Vol. LXXVII of the *Trans.* A. S. C. E. (December, 1914), Vol. XIII of the *Proceedings* of the American Society for Testing Materials (1913), and Vol. II of Hool's "Reinforced Concrete Construction"; and an abstract of the more important portions of it appeared in *Engineering News* of February 6, 1913. It is undoubtedly at present the highest authority in America on the design of reinforced concrete. It covers a good many practical considerations concerning the uses, component materials, and construction of reinforced concrete, and treats thoroughly the methods of design.

The Canadian Society of Civil Engineers adopted at the 1915 Annual Meeting a "Standard General Specification for Concrete and Reinforced Concrete." The provisions of this specification are in close agreement with the 1913 Report of the Joint Committee.

The Engineering Experiment Station of the University of Illinois conducts every year, under the direction of Prof. A. N. Talbot, a large number of valuable experiments on reinforced concrete, the results of which are published from time to time in the bulletins of the university. These bulletins will be mailed to any one desiring them; and every bridge engineer should avail himself of the privilege, for the tests reported therein are among the most important that are being conducted at present.

As the theory of reinforced concrete is well handled in a number of text-books, little space will be devoted to it in this treatise. The two books which have been used most extensively in the author's office are "Concrete, Plain and Reinforced," by Taylor and Thompson, and "Principles of Reinforced Concrete Construction," by Turneaure and Maurer. The former contains, in addition to the theory, a great deal of valuable information regarding the materials of which concrete is made, proper methods of testing and proportioning, etc.; while the latter gives the more complete treatment of the theory of the subject. There is no disagreement of any importance between the two works. Hool's "Reinforced Concrete Construction" has also been utilized to some extent of late. It presents a very clear exposition of the subject, especially in regard to the distribution of internal stresses in beams. Much of its theory has been taken directly from the two books previously mentioned, as is stated in the preface. While the three texts contain a great deal that is in common, the engineer who desires to have a full knowledge of the subject will do well to read all of them, as certain phases thereof are treated from different points of view by the three writers. Any one of them, however, if carefully studied, will give the reader a very good working knowledge of the methods of design.

Several other texts might be mentioned, some of which in many respects are as good as the three discussed above. "Concrete-Steel Construction," a translation by E. P. Goodrich, Esq., C. E., of Prof. Emil Mörsch's "Der Eisenbetonbau," should be especially noted, not only on account of its excellent presentation of the theory of the subject, but also because it gives a good treatment of European methods of design and construction. It contains the "Preliminary Recommendations (Leitsätze) for the Design, Construction, and Testing of Reinforced Concrete Structures," prepared in 1904 by the Verband Deutscher Architekten und Ingenieur Verein and the Deutscher Beton Verein; and also the "Regulations of the Royal Prussian Ministry of Public Works for the Construction of Reinforced Concrete Buildings," dated May 24, 1907.

The page references in this chapter to Turneaure and Maurer's "Principles of Reinforced Concrete Construction" are to the 1909 edition; those to Hool's "Reinforced Concrete Construction" are to the first edition; and those to "Concrete, Plain and Reinforced," by Taylor and Thompson, to the 1909 edition.

Attention is called to the fact that the treatment of reinforced concrete retaining walls is given in Chapter XLIII.

FUNDAMENTAL ASSUMPTIONS IN DESIGNING

Since the stresses in a reinforced concrete member depend upon the mutual actions of the steel and concrete, it is necessary for designing purposes to make a number of assumptions regarding the said actions. The "Specifications for Design" given at the close of this chapter enumerate the assumptions which are to be used, in nearly the same form as they are presented in the 1913 Report of the Joint Committee. Practically identical assumptions are made by all of the other authorities quoted above (including "Der Eisenbetonbau"); and the author sees no good reason for not agreeing with them in every respect. It is true that some of them are not exactly in accordance with the facts, but the resulting unit stresses are not much affected thereby, any appreciable errors being on the side of safety; and as they make the analysis simpler, it is better to adopt them. A very good discussion of the various assumptions that are used more or less at present is to be found in Turneaure and Maurer's "Principles of Reinforced Concrete Construction."

NOTATION

The notation to be used in the various portions of this chapter is collected here for ready reference. The "Standard Notation" of the 1913 Report of the Joint Committee has been followed, with such additions as were found necessary. The symbols given under the heading "Rectangular Beams" will, in general, be used throughout the chapter; but the others will be employed only in the connection for which they are defined.

Rectangular Beams

(See Fig. 37*a*)

E_s = modulus of elasticity of steel.

E_c = modulus of elasticity of concrete,

$n = \frac{E_s}{E_c}$,

b = breadth of beam,

d = depth of beam, from compression face to centre of steel,

A_s = area of tensile steel,

A_i = area of one inclined bar,

A_v = area of one vertical bar in a vertical stirrup,

p = steel ratio for tensile steel $= \frac{A_s}{bd}$,

o = perimeter of one bar in tensile reinforcement,

Σo = sum of perimeters of bars in tensile reinforcement,

m_i = number of inclined bars in one row,

m_v = number of vertical bars in one vertical stirrup,

s_i = horizontal spacing of rows of inclined bars,

s_v = spacing of vertical stirrups,

k = ratio of distance of neutral axis from compression face to depth d,

z = distance of resultant compression from compression face,

j = ratio of lever arm of resisting couple to depth d,

R_s = "coefficient of resistance" relative to the steel,

R_c = "coefficient of resistance" relative to the concrete,

R = "coefficient of resistance" in general,

M_s = moment of resistance relative to the steel,

M_c = moment of resistance relative to the concrete,

M = bending moment, or moment of resistance in general,

V = total shear on section,

C = total compressive stress in concrete,

T = total tensile stress in steel,

f_s = unit tensile or compressive stress in steel,

f_c = maximum unit compressive stress in concrete,

v = maximum unit shear on cross-section,

u = unit bond stress,

V_c = amount of shear taken by concrete,

V_w = amount of shear taken by web reinforcement,

V_i = amount of shear taken by inclined bars,

V_v = amount of shear taken by vertical stirrups,

v_c = portion of maximum unit shear v taken by concrete,

v_w = portion of maximum unit shear v taken by web reinforcement,

v_i = portion of maximum unit shear v taken by inclined bars,

v_v = portion of maximum unit shear v taken by vertical stirrups,

P_i = stress in one inclined bar,

P_v = stress in one vertical bar of a vertical stirrup.

Rectangular Beams Reinforced for Compression

(See Fig. 37*d*)

d' = distance from compression face to centre of compressive steel,
A'_s = area of compressive steel,
p' = steel ratio for compressive steel $= \frac{A'_s}{bd}$,
p_c = tensile steel ratio required when no compressive reinforcement is used and the values of f_c and f_s are both the critical ones (as, for example, 600 and 16,000),
C' = total stress in compressive steel,
f'_s = unit stress in compressive steel.

T-Beams

(See Fig. 37*g*)

b = width of flange,
b' = width of stem,
t = thickness of flange,
p = steel ratio for tensile steel $= \frac{A_s}{bd}$.

Rectangular Beams of Varying Depth

(See Fig. 37*i*)

β = angle between compression face and a normal to the direction of ΣP,
β' = angle between reinforcement and a normal to the direction of ΣP,
p = steel ratio for tensile steel $= \frac{A_s}{bd}$,
$p_1 = \frac{p \cos \beta'}{\cos^2 \beta}$,
$R'_s = \frac{R_s}{\cos^2 \beta}$,
$R'_c = \frac{R_c}{\cos^2 \beta}$,
V_1 = amount of shear carried by concrete and web reinforcement.

Columns under Direct Stress Only

A = total area of cross section for column without hooping, or area enclosed by hoops for hooped columns,
A_c = total area of concrete for column without hooping, or area enclosed by hoops for hooped columns,
A_l = area of longitudinal steel,

p_l = steel ratio for longitudinal steel $= \frac{A_l}{A}$,

P = total direct load, or total safe direct load,

f = average unit stress for entire section $= \frac{P}{A}$.

Rectangular Beams and Columns under Flexure and Direct Stress, with Reinforcement in Tension Face Only. Tension on Part of Section

(See Fig. 37*l*)

P = normal component of resultant force acting on section. To be considered as positive when P is compressive, and negative when P is tensile,

e' = distance from centre of steel to line of action of P. To be considered as positive when P is compressive and lies on the same side of the steel as C; negative when P is tensile and lies on the opposite side of the steel from C,

M' = moment of P about the centre of the steel $= Pe'$.

Rectangular Beams and Columns under Flexure and Direct Stress, with Reinforcement in both Faces

(See Figs. 37*n* and 37*p*)

h = total depth of section,

d' = distance of compressive steel from compression face,

a = distance from steel to centre of section for symmetrical reinforcement,

c = distance from compression face to centroid of transformed section ($= \frac{h}{2}$ for symmetrical reinforcement),

A'_s = area of steel near compression face,

p = steel ratio for tensile steel $= \frac{A_s}{bh}$,

p' = steel ratio for compressive steel $= \frac{A'_s}{bh}$,

A_t = area of transformed section,

I_c = moment of inertia of concrete about centroid of transformed section,

I_s = moment of inertia of steel about centroid of transformed section.

I_t = moment of inertia of transformed section about the centroid thereof,

k = ratio of distance of neutral axis from compression face to depth h,

P = normal component of resultant force acting on the section,

e = eccentricity of P, or distance from the centre of the section to the line of action of P,

e_t = distance from the centroid of the transformed section to the line of action of P,
M = moment of P about the centre of the section = Pe,
M_t = moment of P about the centroid of the transformed section = Pe_t,
f'_s = unit stress in steel near compression face,
f'_c = minimum unit compressive stress in concrete,
f = average stress over entire section = $\frac{P}{bh}$.

Moments of Inertia of Beams, Columns, and Arch Ribs

α = coefficient in the equation $I = \alpha bd^3$.

The Calculation of Stresses in Arch Ribs with Fixed Ends

(See Fig. 37*hh*)

l = length of span of arch,
r = rise of arch,
x, y = co-ordinates of any point with reference to the crown C as an origin, y being positive when measured downward, and x being positive in each half when measured from the crown toward the springing,
α = angle of inclination of the axis at any point,
β = angle of inclination of the axis at the springing,
L = length of rib, measured along the axis $\left(= 2\int ds\right)$,
b = width of rib at any point,
b_o = width of rib at crown,
b_s = width of rib at springing,
h = thickness of rib at any point, measured normally to axis,
h_o = thickness of rib at crown, measured normally to axis,
h_s = thickness of rib at springing, measured normally to axis,
A = area of rib at any point, or area of concrete plus n times area of steel,
A_o = area of rib at crown, or area of concrete plus n times area of steel,
I = moment of inertia of rib at any point, or moment of inertia of concrete plus n times moment of inertia of steel,
I_o = moment of inertia of rib at crown, or moment of inertia of concrete plus n times moment of inertia of steel,
I_s = moment of inertia of rib at springing, or moment of inertia of concrete plus n times moment of inertia of steel,
E = coefficient of elasticity of the concrete, to be taken as

2,000,000 when dimensions are in inches, and as 288,000 000 when they are in feet,

ω = coefficient of linear expansion of concrete (= 0.000006),

t = change of temperature in degrees Fahrenheit, positive when the temperature rises, negative when it falls,

P_1, P_2, P_3, etc., = any loads on the arch, acting in any direction,

p_o = equivalent uniform load at crown,

p_s = equivalent uniform load at springing,

$u = \frac{p_s}{p_o}$,

p = equivalent uniform load at any point,

H_o, V_o, M_o = thrust, shear, and moment at crown, positive when acting in the directions indicated in Fig. 37*hh*,

H_a, M_a = thrust and moment at crown from arch shortening,

H_t, M_t = thrust and moment at crown from temperature change,

C_t = coefficient of temperature thrust in Equation 193,

y_o = vertical distance from crown to plane of contraflexure for arch shortening and temperature change,

T_l = normal thrust at any section in left half of rib, positive when compressive,

T_r = similar quantity for right half of rib,

T = normal thrust in general,

T_s = normal thrust at the springing,

S_l = shear at any section in left half of rib, positive when producing shearing stress in the same direction as V_o in Fig. 37*hh*,

S_r = similar quantity for right half of rib,

V_l = vertical component of T_l at any point in left half of rib positive when acting in the same direction as V_o in Fig. 37*hh*,

V_r = similar quantity for right half of rib,

V_s = vertical component of the thrust at the springing,

V'_l = sum of vertical components of loads on rib between crown and any section in left half of rib, positive when acting downward,

V'_r = similar quantity for right half of rib,

H'_l = sum of horizontal components of loads on rib between crown and any section in left half of rib, positive when acting in the same direction as H_o in Fig. 37*hh*,

H'_r = similar quantity for right half of rib,

M_l = actual bending moment at any section in left half of rib, positive when causing compression in upper fibres,

M_r = similar quantity for right half of rib,

M = actual bending moment in general,

M_s = actual bending moment at springing,

C_m = moment coefficient in Equation 186,

M'_l = moment at any section in the left half of rib due to *external* loads on the said half between the crown and the section considered (H_o, V_o, and M_o being removed), positive when causing compression in the upper fibres,

M'_r = similar quantity for right half of rib,

N_s = number of equal parts into which each half of the horizontal projection of the rib is divided for writing equation of rib,

N = number of any division point, that of the crown being taken as zero,

$\dfrac{N_s}{2}$ = number of equal parts into which each half of the horizontal projection of rib is divided for integration purposes,

ds = length of a division of the arch rib for integration, measured along the axis,

$$\int ds,\ \int\frac{ds}{I},\ \int\frac{yds}{I},\ \int\frac{y^2\,ds}{I},\ \int\frac{xds}{I},\ \int\frac{x^2ds}{I},\ \int\frac{\sec\alpha\ ds}{A},\ \int\frac{M'_l\,ds}{I},$$

$$\int\frac{M'_r\,ds}{I},\ \int\frac{M'_l\,yds}{I},\ \int\frac{M'_r\,yds}{I},\ \int\frac{M'_l\,xds}{I},\ \int\frac{M'_rxds}{I}$$

= summations or integrations taken for one-half of rib only,

$$\int\frac{M'\,ds}{I}=\int\frac{M'_l\,ds}{I}+\int\frac{M'_r\,ds}{I},$$

$$\int\frac{M'yds}{I}=\int\frac{M'_l\,yds}{I}+\int\frac{M'_r\,yds}{I},$$

$$\int\frac{M'xds}{I}=\int\frac{M'_l\,xds}{I}+\int\frac{M'_rxds}{I}.$$

FORMULÆ AND DIAGRAMS FOR DESIGNING BEAMS AND COLUMNS

In the following pages there are given various formulæ and diagrams for designing reinforced concrete members. Some of these have already appeared in the Report of the Joint Committee, in various text books, and in technical papers; while others have not been published hitherto, so far as the author knows. They have been grouped together for the convenience of the busy engineer. A brief discussion, covering the derivation of the new formulæ and diagrams and the sources from which the others have been taken, will first be given; and this should be read in conjunction with the study of the formulæ and diagrams themselves.

The derivation of the formulæ for rectangular beams can be found in any of the text-books on reinforced concrete. Fig. 37*b* is the well-known French diagram, first published by A. W. French, Esq., M. Am. Soc. C. E., in the *Trans.* Am. Soc. C. E., Vol. LVI. The method of plotting Fig. 37*c* needs no explanation.

The formulæ for rectangular beams reinforced for compression are standard. Fig. 37*e* was adapted from a diagram given by Samuel Klein, Esq., C. E., in *Engineering Record* of August 30, 1913, but is believed to be a decided improvement thereon. Figs. 37*e'* and 37*f* were worked up from the curves of Fig. 37*e*. The formulæ assume that the use of compression steel adds n times its area to the section, rather than $n - 1$ times its area, which is the correct amount. This introduces errors of about one or two per cent on the danger side in the concrete stresses. As no material simplification of the formulæ is obtained by using n rather than $n - 1$, it is difficult to see why the practice has been followed; but the author has concluded to adopt the formulæ universally used, as the errors are negligible. Exact results can be obtained by entering the various diagrams with a value of p' equal to $\frac{14}{15}$ of that employed in the actual beam.

The formulæ given for T-beams are standard. Fig. 37*h* is similar in arrangement to Fig. 37*e*; and Fig. 37*h'* was worked up from the curves of Fig. 37*h*.

Formulæ for rectangular beams of varying depth are developed in articles by W. Cain, Esq., M. Am. Soc. C. E., entitled "Stresses in Wedge-Shaped Reinforced Concrete Beams," which were published in Vol. LXXVII of the *Trans.* Am Soc. C. E., and in the *Proc.* Am. Soc. C. E. of December, 1914. However, Equations 85, 86, and 139 had been developed independently in the author's office several years earlier by E. A. Slettum, Esq., C. E. The quantities p_1, R'_s, and R'_c are introduced in order to facilitate the plotting of Fig. 37*j*, which was drawn up specially for this treatise. The lower right-hand portion of this diagram is a graphical representation of Equations 82 and 84, and is evidently the same as Fig. 37*b*. The lower left-hand portion expresses the equations defining R'_s and R'_c in terms of R_s, R_c, and β; and the two upper portions, the equation defining p_1 in terms of p, β, and β'.

The formulæ for columns under direct stress only are standard. The method of drawing Fig. 37*k* requires no explanation.

The equations for the design of beams under flexure and direct stress with reinforcement in one face only have not been published previously, so far as the author knows, and it is therefore necessary to give the derivation for them here. See Fig. 37*l*.

As in the case of beams under flexure only, we may evidently write

$$\frac{f_s}{nf_c} = \frac{1-k}{k}, \qquad \text{[Eq. 1]}$$

which is the same as Equation 100, but is in a different form. Since the algebraic sum of the compressive and tensile forces must equal P, we have the equation

$$\frac{1}{2}f_c bkd - f_s pbd = P; \qquad \text{[Eq. 2]}$$

and since the moment of P about the centre of the reinforcement must be balanced by the moment of the compressive stresses on the section about the same point, we may write

$$\frac{1}{2}f_c bkd\,(1 - \frac{k}{3})\,d = Pe' = M'. \qquad \text{[Eq. 3]}$$

The product Pe' is always positive, since P will always have the same sign as e'. (See Notation.) By substituting the value of f_s from Equation 1 into Equation 2, we have

$$\frac{1}{2}f_c bkd - nf_c pbd\,\frac{1-k}{k} = P; \qquad \text{[Eq. 4]}$$

and on dividing this latter equation by Equation 3, we find

$$\frac{1}{e'} = \frac{\frac{1}{2}k - np\,\frac{1-k}{k}}{\frac{1}{2}k\left(1 - \frac{k}{3}\right)d}. \qquad \text{[Eq. 5]}$$

This latter equation may be put into the form

$$\frac{k^2 - 2np\,(1-k)}{k^2\left(1 - \frac{k}{3}\right)} = \frac{d}{e'}, \qquad \text{[Eq. 6]}$$

which is the same as Equation 96; and Equation 3 may be written

$$M' = \frac{1}{2}f_c k\left(1 - \frac{k}{3}\right)bd^2 = \frac{1}{2}f_c kjbd^2, \qquad \text{[Eq. 7]}$$

which is Equation 99.

In order to draw convenient working diagrams, we put

$$\frac{1}{2}f_c kj = R_c. \qquad \text{[Eq. 8]}$$

The lower portion of Fig. 37*m* is a graphical representation of this formula; and to it there have been added curves for f_s, drawn by means of Equation 1. The upper portion of Fig. 37*m* is a graphical representation of Equation 6, which, as previously stated, is the same as Equation 96.

The device of taking moments about the centre of the steel was proposed by C. W. Yelm, Esq., C.E., who also derived Equations 6 and 7. Fig. 37*m* was worked up directly for this treatise.

The equations for beams and columns under flexure and direct stress with reinforcement in both faces are similar to those given in Turneaure and Maurer's "Principles of Reinforced Concrete Construction," to which book the reader is referred for their derivation. In Equations 113 and 114 of Case II there have also been given the values of f_c and f'_c in terms of the direct load, as the use of the expression involving the moment does not give accurate results for small values of $\frac{e}{h}$ unless the calculations are carried out to several significant figures. In these expressions, the quantity $\frac{P}{bh} \cdot \frac{1}{1 + 2np}$ is the unit stress due to direct load, and $\frac{P}{bh} \cdot 6\frac{e}{h} \cdot \frac{1}{1 + 2np\frac{a^2}{h^2}}$ is that due to the moment. The derivation of these two quantities needs no explanation. The full lines of Fig. 37*o* were drawn by means of Equations 113 and 114 for three values of $\frac{d'}{h}$; and the dotted lines on this same figure were derived from the curves of Fig. 37*q*. The latter diagram applies for the same three values of $\frac{d'}{h}$ as does Fig. 37*o*. The full lines of Fig. 37*q* give the same information as is given by Plate XIV of Turneaure and Maurer's book, and the dotted lines the same as by Plate XIII of that text; but the diagrams are drawn for values of $\frac{h}{e}$ rather than $\frac{e}{h}$, thus keeping the lines for values of p from running too close together for small values of $\frac{e}{h}$, and at the same time making it possible to include on the drawing all values of $\frac{e}{h}$ ranging from 0.1 to infinity (since $\frac{h}{e}$ is made to vary from 10 to 0). It will be noted that the diagrams of Figs. 37*o* and 37*q* overlap, there being a considerable range of values of $\frac{e}{h}$ — from about $\frac{1}{7}$ to $\frac{1}{4}$ — for which either figure is applicable for all plotted values of p. The comment made previously, when referring to double reinforced beams, concerning the use of n times the area of the compressive steel rather than $n - 1$ times the area thereof, applies to the formulæ and diagrams for beams and columns under flexure and direct stress with reinforcement in both faces; and the error can be allowed for in the same manner.

The formulæ for the calculation of unit shearing and bond stresses are standard. A very complete study of bond stresses was made by

Duff A. Abrams, Esq., C.E., at the University of Illinois, and the report was published in Bulletin No. 71 of that institution.

The formulæ given for the design of web reinforcement are standard. They should be recognized as being more or less tentative, as our knowledge of the action of such reinforcement is still incomplete. The formulæ assume that all web reinforcement is fully developed at the tension reinforcement and also within the compression area, so that the full tensile strength thereof is available from the tension reinforcement to the neutral axis. Fig. 37*r* was plotted by means of Equations 130 and 135.

Rectangular Beams

Position of neutral axis,

$$k = \sqrt{2pn + (pn)^2} - pn = \frac{n f_c}{f_s + n f_c}, \qquad \text{[Eq. 9]}$$

$$= \tfrac{3}{8} \text{ roughly.} \qquad \text{[Eq. 10]}$$

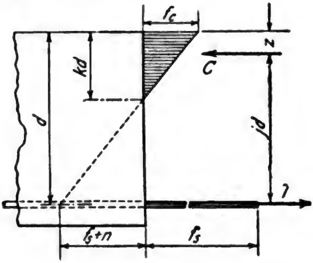


Fig. 37*a*.

Arm of resisting couple,

$$j = 1 - \tfrac{1}{3} k, \qquad \text{[Eq. 11]}$$

$$= \tfrac{7}{8} \text{ approximately.} \qquad \text{[Eq. 12]}$$

Coefficients of resistance,

$$R_s = f_s p j, \qquad \text{[Eq. 13]}$$

$$R_c = \tfrac{1}{2} f_c k j. \qquad \text{[Eq. 14]}$$

Moments of resistance,

$$M_s = f_s p j b d^2 = R_s b d^2, \qquad \text{[Eq. 15]}$$

$$= f_s A_s \tfrac{7}{8} d \text{ approximately,} \qquad \text{[Eq. 16]}$$

$$M_c = \tfrac{1}{2} f_c k j b d^2 = R_c b d^2, \qquad \text{[Eq. 17]}$$

$$= \tfrac{1}{6} f_c b d^2 \text{ roughly.} \qquad \text{[Eq. 18]}$$

Fibre stresses,

$$f_s = \frac{M}{A_s j d} = \frac{M}{p j b d^2} = \frac{k f_c}{2 p}, \qquad \text{[Eq. 19]}$$

$$= \frac{M}{\frac{7}{8}\,d\,A_s} \text{ approximately,} \qquad \text{[Eq. 20]}$$

$$f_c = \frac{2\,M}{j\,k\,b\,d^2} = \frac{2\,p\,f_s}{k}, \qquad \text{[Eq. 21]}$$

$$= \frac{6\ M}{b\,d^2} \text{ roughly.} \qquad \text{[Eq. 22]}$$

Steel ratio,

$$p = \frac{k\,f_c}{2 f_s} = \frac{1}{2\frac{f_s}{f_c}\left(\frac{f_s}{n\,f_c}+1\right)} = \frac{k^2}{2\,n\,(1-k)} = \frac{A_s}{bd} \qquad \text{[Eq. 23]}$$

$$= \frac{3}{16}\frac{f_c}{f_s} \text{ roughly.} \qquad \text{[Eq. 24]}$$

Cross-section of beam for given bending moment M,

$$b\,d^2 = \frac{M}{f_s\,p\,j} = \frac{M}{R_s}, \qquad \text{[Eq. 25]}$$

$$= \frac{2\ M}{f_c\,k\,j} = \frac{M}{R_c}, \qquad \text{[Eq. 26]}$$

$$= \frac{6\,M}{f_c} \text{ roughly.} \qquad \text{[Eq. 27]}$$

Steel area,

$$A_s = \frac{M}{f_s\,j\,d} = p\,b\,d, \qquad \text{[Eq. 28]}$$

$$= \frac{M}{\frac{7}{8}\,d\,f_s} \text{ approximately.} \qquad \text{[Eq. 29]}$$

Equations 12, 16, 20, and 29 give results that are sufficiently accurate in nearly all cases. For small values of p, the value of j approaches unity, as can be seen from Fig. 37*b*. Equations 10, 18, 22, 24, and 27 are to be used in making rough calculations only.

Figs. 37*b* and 37*c* are drawn for $n = 15$. The curves in the lower portion of Fig. 37*b* give simultaneous values of f_s, f_c, p, R_s, and R_c (both of the latter being called R); and those in the upper portion record corresponding values of k and j. The curves of Fig. 37*c* are drawn on the assumption that f_c does not exceed 600 pounds per square inch, and that f_s is not greater than 16,000 pounds per square inch. The upper portion of this figure gives simultaneous values of the depths and the steel areas per foot of width which are required for any given bending moment; and

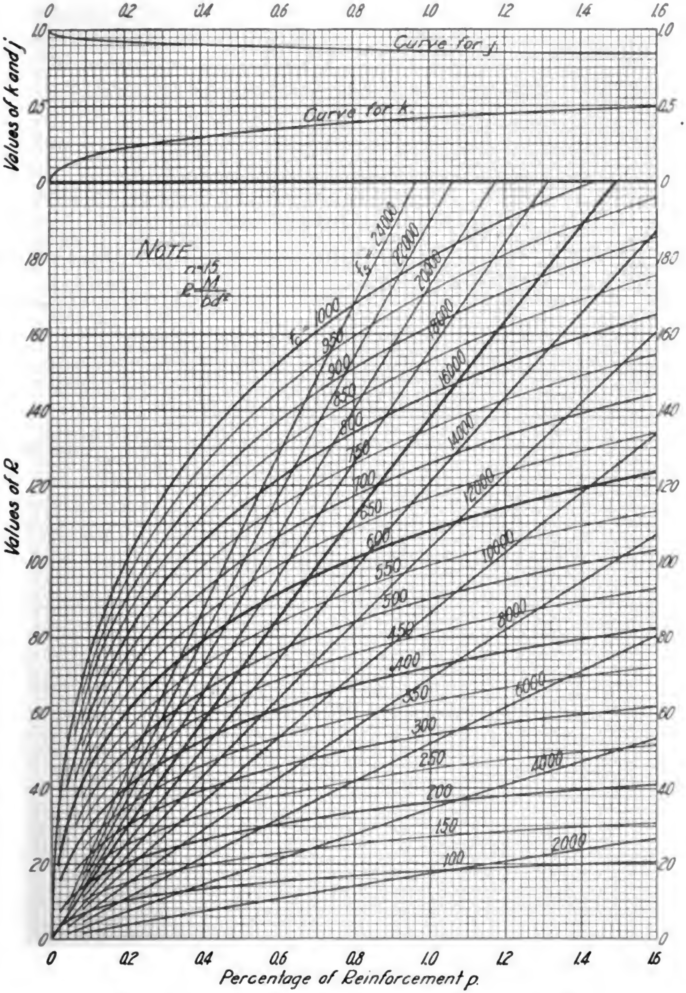


Fig. 37*b*. Diagram for the Design of Rectangular Beams.

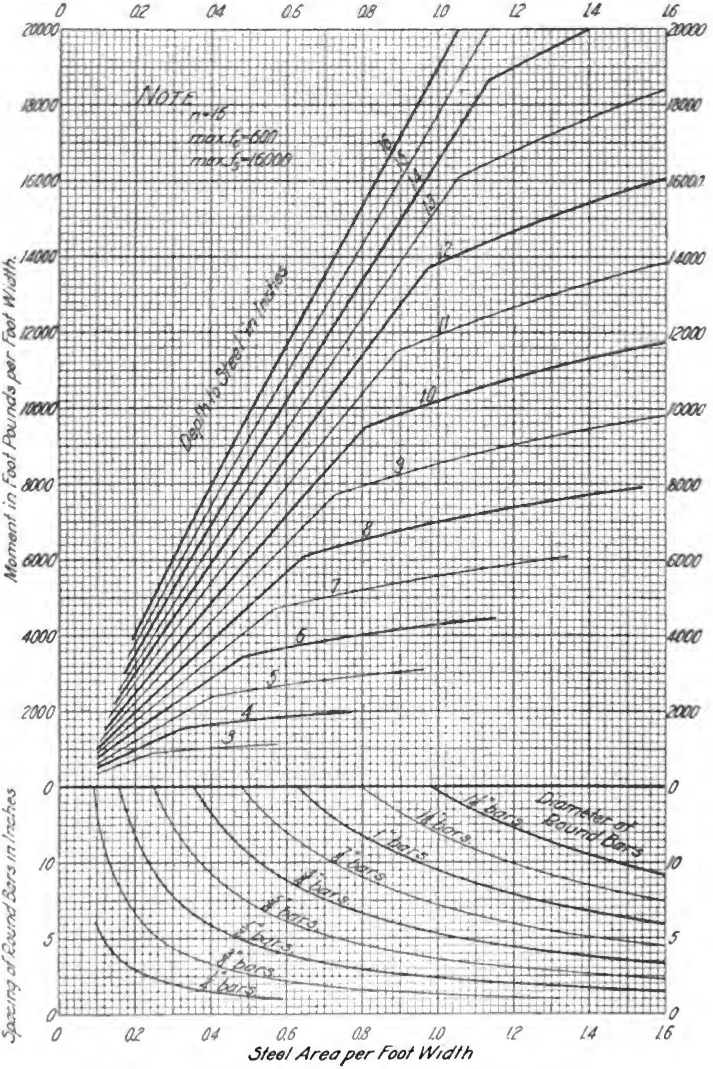


FIG. 37c. Diagram for the Design of Slabs and Small Beams.

the lower portion shows the steel areas per foot of width furnished by round bars of various diameters and spacings. The diagram is arranged so that it is possible to trace from one portion to the other. The upper portion will serve for slabs and small beams only; but the lower portion can frequently be used in conjunction with Fig. 37*b*, even when the upper portion is not applicable. The curves of Figs. 37*b* and 37*c* are so well known that no explanation of their application is necessary.

For ordinary design work, the calculator should keep in mind the values of R and p which correspond to the values of f_c and f_s which he employs (as, for instance, $R = 95$ and $p = 0.68$ per cent when $f_c = 600$ and $f_s = 16{,}000$), and thus avoid the necessity of constantly keeping the diagrams at hand. Also, it is a good plan to figure the steel areas by Equation 29 or the more exact formula $\frac{M}{f_s j\,d}$, rather than by the expression $p\,b\,d$. One is then less likely to make large errors; and, when a beam is known to be under-reinforced, a considerable amount of time is saved, as it is then unnecessary to figure the value of b. When f_s equals 16,000, $\frac{M}{\frac{7}{8}\,d f_s}$ becomes $\frac{M}{14{,}000\,d}$.

The following table of areas and weights of plain round and square bars will be found convenient. Deformed bars will usually weigh about one per cent more than plain bars.

Diameter of Round Bar or Side of Square Bar Inches	Round Bars		Square Bars	
	Area Square Inches	Weight Pounds per Lineal Foot	Area Square Inches	Weight Pounds per Lineal Foot
¼	0.049	0.17	0.063	0.21
⅜	0.110	0.38	0.141	0.48
½	0.196	0.67	0.25	0.85
⅝	0.31	1.04	0.39	1.33
¾	0.44	1.50	0.56	1.91
⅞	0.60	2.04	0.76	2.60
1	0.78	2.67	1.00	3.40
1⅛	0.99	3.38	1.26	4.30
1¼	1.23	4.17	1.56	5.31
1⅜	1.48	5.05	1.89	6.43
1½	1.77	6.01	2.25	7.65

Rectangular Beams Reinforced For Compression

Position of neutral axis,

$$k = \sqrt{2\,n\left(p + p'\,\frac{d'}{d}\right) + n^2\,(p + p')^2} - n\,(p + p') = \frac{n\,f_c}{f_s + n\,f_c} \qquad \text{[Eq. 30]}$$

$$= \tfrac{3}{8} \text{ roughly.} \qquad \text{[Eq. 31]}$$

Position of resultant compression,

$$z = \frac{\frac{1}{3}k^3 d + 2p'nd'\left(k - \frac{d'}{d}\right)}{k^2 + 2p'n\left(k - \frac{d'}{d}\right)}, \qquad \text{[Eq. 32]}$$

$$= \tfrac{1}{8} \text{ approximately.} \qquad \text{[Eq. 33]}$$

Arm of resisting couple,

$$jd = d - z, \qquad \text{[Eq. 34]}$$

$$= \tfrac{7}{8}\, d \text{ approximately.} \qquad \text{[Eq. 35]}$$

Coefficient of resistance,

$$R_s = f_s p j. \qquad \text{[Eq. 36]}$$

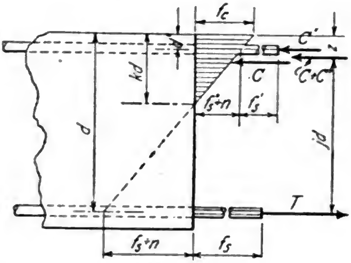


FIG. 37*d*.

Moments of resistance,

$$M_s = f_s p j b d^2 = R_s b d^2, \qquad \text{[Eq. 37]}$$

$$= \tfrac{7}{8} f_s A_s d \text{ approximately,} \qquad \text{[Eq. 38]}$$

$$M_c = \left[k\left(\frac{1}{2} - \frac{1}{6}k\right) + \frac{np'}{k}\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)\right] f_c b d^2, \qquad \text{[Eq. 39]}$$

$$= (0.15 + 10\,p') f_c b d^2 \text{ roughly.} \qquad \text{[Eq. 40]}$$

Fibre stresses,

$$f_s = \frac{M}{A_s j d} = \frac{M}{p j b d^2} = n f_c \frac{1-k}{k}, \qquad \text{[Eq. 41]}$$

$$= \frac{M}{\frac{7}{8} d.\, A_s} \text{ approximately,} \qquad \text{[Eq. 42]}$$

$$f_c = \frac{6M}{b d^2\left[3k - k^2 + \frac{6p'n}{k}\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)\right]} = f_s \frac{k}{n(1-k)}, \qquad \text{[Eq. 43]}$$

$$f_s' = n f_c \frac{\left(k - \frac{d'}{d}\right)}{k} = f_s \frac{k - \frac{d'}{d}}{1 - k}. \qquad \text{[Eq. 44]}$$

Ratio of tensile steel,

$$p = \frac{A_s}{bd'} \qquad \text{[Eq. 45]}$$

$$= \frac{M}{\frac{7}{8}\, b\, d^2 f_s} \text{ approximately.} \qquad \text{[Eq. 46]}$$

Area of tensile steel,

$$A_s = \frac{M}{f_s j\, d} = p\, b\, d, \qquad \text{[Eq. 47]}$$

$$= \frac{M}{\frac{7}{8}\, d\, f_s} \text{ approximately.} \qquad \text{[Eq. 48]}$$

Ratio of compressive steel,

$$\left.\begin{aligned} p' &= 2 \quad (p - p_c) \text{ approximately for } \frac{d'}{d} = 0.05, \\ &= 2.4\, (p - p_c) \text{ approximately for } \frac{d'}{d} = 0.10, \\ &= 3.2\, (p - p_c) \text{ approximately for } \frac{d'}{d} = 0.15. \end{aligned}\right\} \qquad \text{[Eq. 49]}$$

Figs. 37*e*, 37*e′*, and 37*f* can be used for the design of double-reinforced beams; and Equations 38, 42, 46, 48, and 49 afford an approximate solution which is nearly enough correct for ordinary cases.

Of the three diagrams, Fig. 37*e* is the most general in its application. It is drawn for $n = 15$. The central left-hand diagram gives simultaneous values of k, f_s, and f_c; the central right-hand diagram shows simultaneous values of pj, R_s, and f_s; the upper and lower left-hand portions indicate simultaneous values of k, p, and p' for three different values of $\frac{d'}{d}$; and the upper and lower right-hand portions present simultaneous values of pj, p, and p' for the same values of $\frac{d'}{d}$. The diagrams are arranged in such a manner that it is convenient to trace from one to the other. Curves for the values of f'_s could be added to the central left-hand diagram; but they would be of no value, since the value of f'_s can never be excessive. For a value of $\frac{d'}{d}$ differing but slightly from one of the three plotted values, the curves for this plotted value can be used; but for any other $\frac{d'}{d}$ it will be necessary to figure for two values and interpolate, or else use the nearest one and then estimate the effect of the difference in the $\frac{d'}{d}$'s by means of Fig. 37*f*.

As the employment of Fig. 37*e* for the design of beams is somewhat tedious, an auxiliary diagram, Fig. 37*e′*, has been prepared. It applies

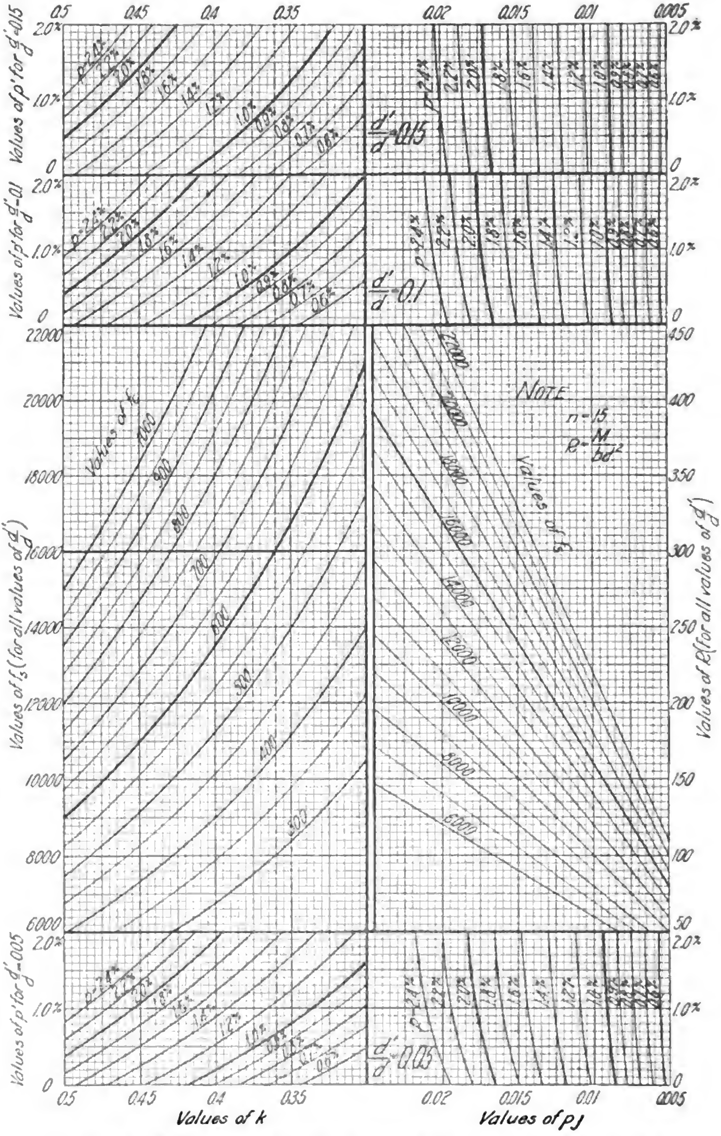


FIG. 37e. Diagram for the Design of Double-reinforced Beams in General.

only when $n = 15$, $f_c = 600$, and $f_s = 16{,}000$; but a similar diagram can be easily prepared for any other set of values of f_s and f_c. In general, it cannot be employed to find the unit stresses in a beam which has already

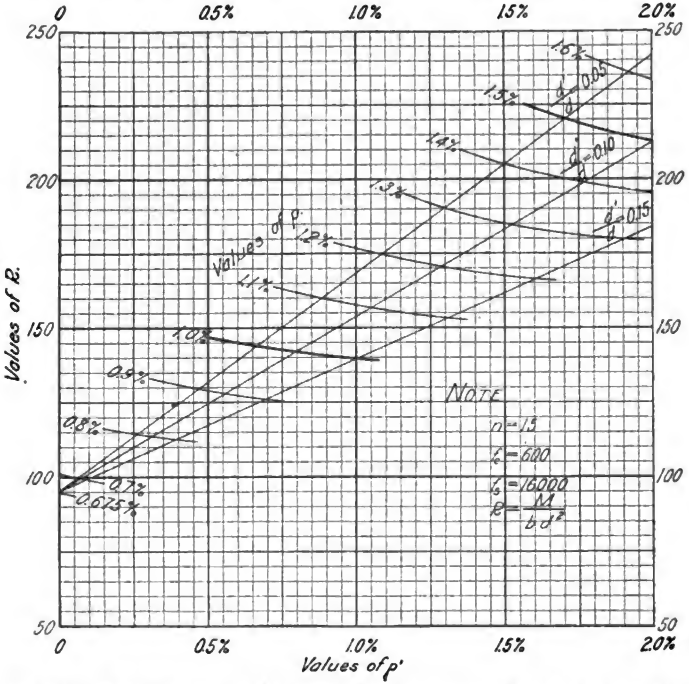


FIG. 37e'. Special Diagram for the Design of Double-reinforced Beams when $f_c = 600$ and $f_s = 16{,}000$.

been designed. It gives simultaneous values of R, p, and p', and can be used directly for any value of $\frac{d'}{d}$.

Fig. 37f was also drawn up for $n = 15$, $f_c = 600$, and $f_s = 16{,}000$; but it applies with very small error for any ordinary combination of f_c and f_s. It gives directly the percentage changes in the steel and concrete stresses caused by the addition of a given percentage of compressive steel to a beam with reinforcement in the tension face only. It is to be used in connection with Fig. 37b.

Equations 38, 42, 46, 48, and 49 afford a means of designing a beam when no diagrams are at hand. It is necessary to know the value of p_c

and the corresponding value of R (as, for $f_c = 600$ and $f_s = 16{,}000$, $p_c =$ 0.68 per cent and $R = 95$).

The following problems illustrate the use of the foregoing equations and diagrams:

1. Design the reinforcement of a beam having a width of 20″ and a depth to the steel of 40″ to carry a moment of 4,000,000 in.-lbs., with $f_c = 600$ and $f_s = 16{,}000$.

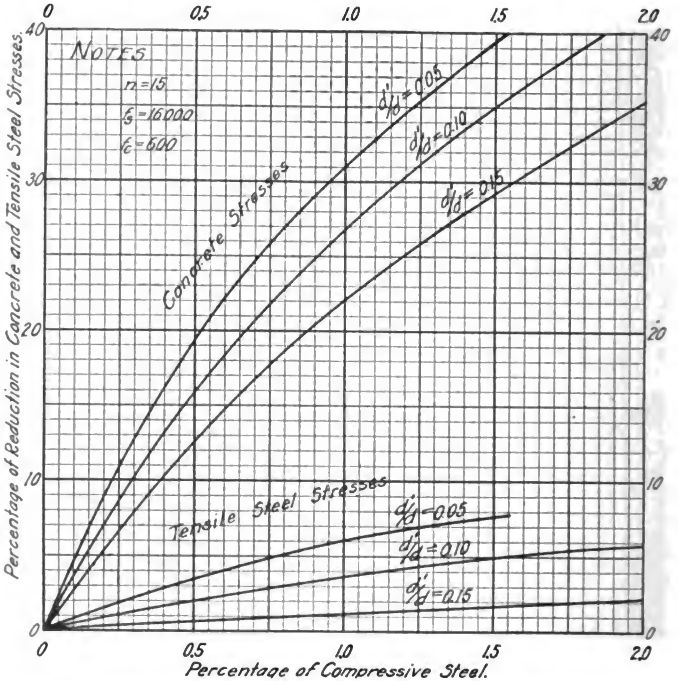


FIG. 37*f*. Percentage of Reduction in Concrete and Steel Stresses Due to Compressive Reinforcement.

We first figure R, finding

$$R = \frac{4{,}000{,}000}{20 \times 40 \times 40} = 125.$$

Entering Fig. 37*b* with $R = 125$, and tracing horizontally to the line for $f_s = 16{,}000$, we find $f_c = 720$ and $p = 0.9\%$, showing that compressive reinforcement is necessary. This will be placed 2″ below the top, so that

$\frac{d'}{d} = \frac{2}{40} = 0.05$. The amount required can be found in any one of four ways.

Employing first Fig. 37*e*, we enter at the right with $R = 125$, trace horizontally to the line for $f_s = 16{,}000$, and then vertically downward to the lower portion of the figure. Since the curves for p are nearly vertical, it is evident that the required value of p is a trifle less than 0.9 per cent. Holding a pencil at the point just found, we then enter the central left-hand portion of the diagram with $f_c = 600$ and $f_s = 16{,}000$, and trace downward till we strike the line $p = 0.9\%$ in the bottom portion of the figure. This indicates $p' = 0.46\%$. Tracing the pencil on the right vertically to this value of p', we find $p = 0.87\%$; and tracing vertically in the left-hand portion to this value of p, we get $p' = 0.4\%$. Evidently it is unnecessary to shift again the pencil on the right, so that the final result is $p = 0.87\%$ and $p' = 0.4\%$.

Next using Fig. 37*e′*, we enter at the left with $R = 125$, and trace horizontally to the line for $\frac{d'}{d} = 0.05$. We then read the value of p as 0.87%, and that of p' as 0.41%.

Employing now Fig. 37*f*, we first compute the percentage of reduction required in the concrete stresses, which is $\frac{720 - 600,}{720}$ or 16 per cent. Entering Fig. 37*f* at the left with 16 per cent, we trace horizontally to the upper line for $\frac{d'}{d} = 0.05$, and then vertically downward to the lower curve for the same value of $\frac{d'}{d}$. This indicates that $p' = 0.4\%$ and that the percentage of reduction in the tensile steel stresses is 3; so that the required value of p is 0.9×0.97, or 0.87%.

Finally, employing Equations 46, 48, and 49, and knowing that $p_e = 0.68\%$, and that the corresponding value of R is 95, we have,

$$A_s = \frac{4{,}000{,}000}{\tfrac{7}{8} \times 40 \times 16{,}000} = 7.14 \text{ sq. ins.},$$

$$p = \frac{7.14}{40 \times 20} = 0.89\%,$$

and

$$p' = 2(0.89 - 0.68) = 0.42\%.$$

2. What are the steel and concrete stresses in a beam which is subjected to a moment of 4,800,000 in.-lbs., the width being 20 inches, the depth to the tensile steel 40 inches, the percentage of tensile steel 1.2, and the percentage of compressive steel 1.0, with $\frac{d'}{d}$ equal to 0.10?

We first find

$$R = \frac{4{,}800{,}000}{20 \times 40 \times 40} = 150.$$

Using Fig. 37*e*, we enter first in the upper right-hand portion with $p = 1.2\%$ and $p' = 1.0\%$, and trace vertically downward to the line for $R = 150$, whence we find $f_s = 14{,}200$. Entering now the upper left-hand portion with the same values of p and p', and then tracing vertically downward to $f_s = 14{,}200$, we get $f_c = 570$.

Again, employing Fig. 37*f*, we first find, from Fig. 37*b*, that f_c equals 790 and f_s equals 14,800. Entering Fig. 37*f* with $p' = 1.0\%$, we find the reduction in the concrete stress to be 27 per cent, and that in the steel stress 4 per cent. We therefore have

$$f_c = 790 \times 0.73 = 580,$$

and

$$f_s = 14{,}800 \times 0.96 = 14{,}200.$$

T-BEAMS

Case I. Neutral Axis in the Flange

The formulæ and diagrams for rectangular beams are to be used. Equation (1) or the curve for k on the upper portion of Fig. 37*b* can be employed to determine the location of the neutral axis.

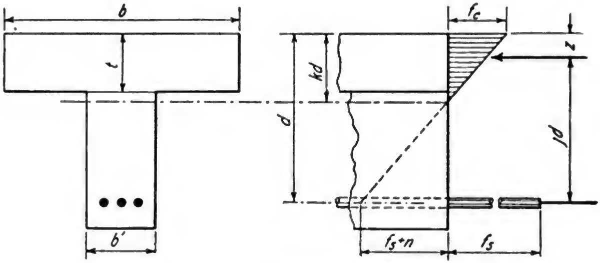


FIG. 37*g*.

Case II. Neutral Axis in the Web: Compression in the Web Neglected

Position of neutral axis,

$$k = \frac{n\,A_s + \tfrac{1}{2}\,b\,t\,\dfrac{t}{d}}{n\,A_s + b\,t} = \frac{p\,n + \tfrac{1}{2}\left(\dfrac{t}{d}\right)^2}{p\,n + \dfrac{t}{d}}, \qquad \text{[Eq. 50]}$$

$$= \tfrac{3}{8} \text{ roughly.} \qquad \text{[Eq. 51]}$$

Position of resultant compression,

$$z = \frac{2k - \frac{4}{3}\frac{t}{d}}{2k - \frac{t}{d}} \cdot \frac{t}{2}, \qquad \text{[Eq. 52]}$$

$$\left.\begin{aligned} &= \frac{t}{2} \text{ when } \frac{t}{d} < 0.25 \\ &= \frac{1}{8} d \text{ when } \frac{t}{d} > 0\ 25 \end{aligned}\right\} \text{approximately.} \qquad \text{[Eq. 53]}$$

Arm of resisting couple,

$$jd = d - z, \qquad \text{[Eq. 54]}$$

$$\left.\begin{aligned} &= d - \frac{t}{2} \text{ when } \frac{t}{d} < 0.25 \\ &= \frac{7}{8} d \text{ when } \frac{t}{d} > 0.25 \end{aligned}\right\} \text{approximately.} \qquad \text{[Eq. 55]}$$

Coefficient of resistance,

$$R_s = f_s\, p\, j. \qquad \text{[Eq. 56]}$$

Moments of resistance,

$$M_s = f_s A_s (d - z) = f_s\, p\, j\, b\, d^2 = R_s\, b\, d^2, \qquad \text{[Eq. 57]}$$

$$\left.\begin{aligned} &= f_s A_s \left(d - \frac{t}{2}\right) \text{ when } \frac{t}{d} < 0.25 \\ &= \frac{7}{8} f_s A_s\, d \quad \text{ when } \frac{t}{d} > 0.25 \end{aligned}\right\} \text{approximately,} \qquad \text{[Eq. 58]}$$

$$M_c = f_c \left(1 - \frac{t}{2\,k\,d}\right) b\, t\, (d - z), \qquad \text{[Eq. 59]}$$

$$= f_c \left(1 - \frac{5}{3}\frac{t}{d}\right) b\, t\, d \text{ approximately.} \qquad \text{[Eq. 60]}$$

Fibre stresses,

$$f_s = \frac{M}{A_s (d - z)} = \frac{M}{A_s\, j\, d}, \qquad \text{[Eq. 61]}$$

$$\left.\begin{aligned} &= \frac{M}{A_s \left(d - \frac{t}{2}\right)} \text{ when } \frac{t}{d} < 0.25 \\ &= \frac{M}{\frac{7}{8} d\, A_s} \quad \text{ when } \frac{t}{d} > 0.25 \end{aligned}\right\} \text{approximately,} \qquad \text{[Eq. 62]}$$

$$f_c = \frac{M}{\left(1 - \frac{t}{2\,k\,d}\right) b\, t\, (d - z)} = f_s \frac{k}{n\,(1 - k)}, \qquad \text{[Eq. 63]}$$

$$= \frac{M}{\left(1 - \frac{5}{3}\frac{t}{d}\right) b\,t\,d} \text{ approximately.} \qquad \text{[Eq. 64]}$$

Steel area,

$$A_s = \frac{M}{f_s\,(d - z)} = \frac{M}{f_s\,j\,d} = p\,b\,d, \qquad \text{[Eq. 65]}$$

$$\left.\begin{aligned} &= \frac{M}{f_s\left(d - \frac{t}{2}\right)} && \text{when } \frac{t}{d} < 0.25 \\ &= \frac{M}{f_s \cdot \frac{7}{8} d} && \text{when } \frac{t}{d} > 0.25 \end{aligned}\right\} \text{approximately.} \qquad \text{[Eq. 66]}$$

Width of flange,

$$b = \frac{M}{f_c\left(1 - \frac{t}{2\,k\,d}\right) t\,(d - z)}, \qquad \text{[Eq. 67]}$$

$$= \frac{M}{f_c\left(1 - \frac{5}{3}\frac{t}{d}\right) t\,d} \text{ approximately.} \qquad \text{[Eq. 68]}$$

Fig 37*h* applies for $n = 15$. The lower left-hand diagram gives simultaneous values of k, f_s, and f_c; the upper left-hand diagram shows simultaneous values of k, $\frac{t}{d}$, and p; the upper right-hand diagram indicates simultaneous values of p, $p\,j$, and $\frac{t}{d}$; and the lower right-hand diagram presents simultaneous values of f_s, $p\,j$, and R_s. The diagrams are arranged in such a manner that it is convenient to trace from one to the other.

As the use of Fig. 37*h* is more or less tedious, an auxiliary diagram, Fig. 37*h'*, has been prepared. It applies only when $n = 15$, $f_c = 600$, and $f_s = 16{,}000$; but similar curves for other values of f_c and f_s can be readily drawn up. It can be employed merely for designing beams, not for computing the stresses in beams already designed. It gives directly the values of R, p, and j for any value of $\frac{t}{d}$. The horizontal portions of the curves at the right correspond to values of $\frac{t}{d}$ greater than k, under which conditions the stresses are the same as for a rectangular beam. For beams in which the flange width must be checked, R can be used for that purpose, and the steel area computed by reading the value of j and using the relationship $\frac{M}{f_s\,j\,d}$, or by reading p and employing the expression $p\,b\,d$.

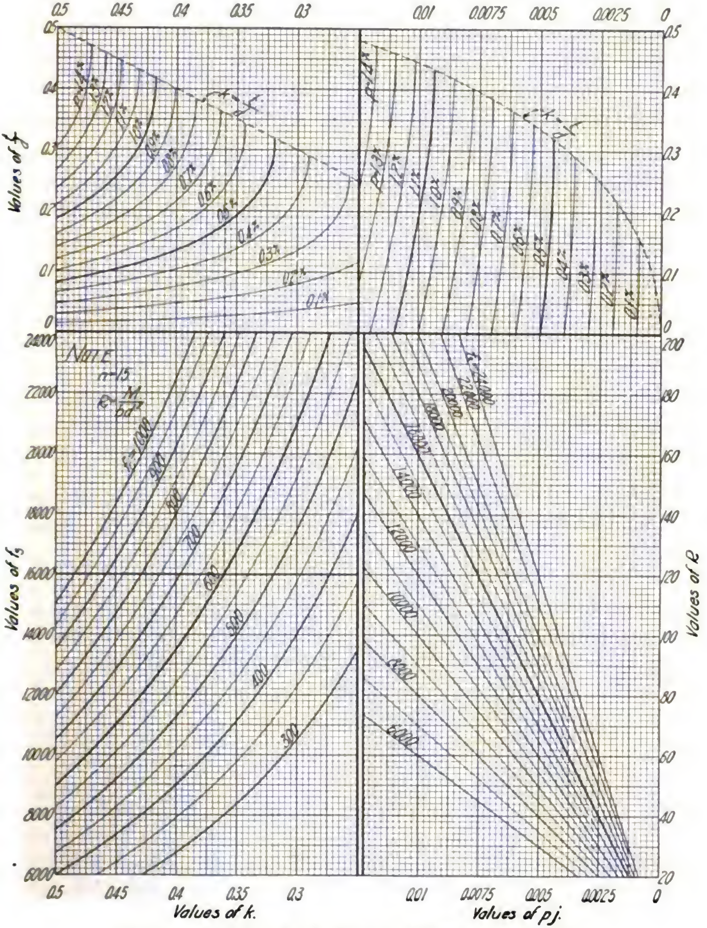


FIG. 37h. Diagram for the Design of T-Beams in General.

The former method is to be preferred, as there is less likelihood of large error. If it is known that the flange width is ample, it is merely necessary to read the value of j and use the expression $\frac{M}{f_s\, j\, d}$.

The curves for rectangular beams can be used with small error unless $\frac{t}{d}$ is considerably smaller than k. The values of f_s obtained in this manner will be a trifle too high, and those of f_c somewhat too low.

Equations 53, 55, 58, 60, 62, 64, 66, and 68 are sufficiently accurate for most cases; and the use thereof is advisable, as it makes one independent of the diagrams.

For a beam in which $\frac{t}{d}$ is small and $\frac{b'}{b}$ is fairly large, the use of the formulæ of Case II will give values of f_s or A_s which are too small, and values of b or f_c which are much too large. Such beams should be considered to come under Case III.

The following problems illustrate the application of Figs. 37*h* and 37*h′*:

1. Design the section of a T-beam for a moment of 10,000,000 inch-pounds, on the assumption that $d = 60''$, $t = 12''$, $f_c = 600$, and $f_s = 16{,}000$.

We shall first employ Fig. 37*h*. Entering the lower left-hand portion of the diagram with $f_c = 600$ and $f_s = 16{,}000$, and tracing vertically upward to $\frac{t}{d} = \frac{12}{60} = 0.2$, we find $p = 0.54\%$. We then project horizontally to this same value of p in the upper right-hand portion, and then trace vertically downward to $f_s = 16{,}000$, giving $R = 79$. We then compute the required width of flange and the steel area as follows:

$$b = \frac{10{,}000{,}000}{79 \times 60 \times 60},$$

$$= 35'',$$

and

$$A_s = 0.0054 \times 35 \times 60 = 11.3 \text{ sq. in.}$$

Using next Fig. 37*h′*, and entering with $\frac{t}{d} = 0.2$, we find $R = 79$, so that b, as before, is found to be 35″. We can then read j as 0.91, whence we have

$$A_s = \frac{10{,}000{,}000}{0.91 \times 60 \times 16{,}000} = 11.5 \text{ sq. in.};$$

or we can read p as 0.54%, whence we find

$$A_s = 0.0054 \times 35 \times 60 = 11.3 \text{ sq. in.}$$

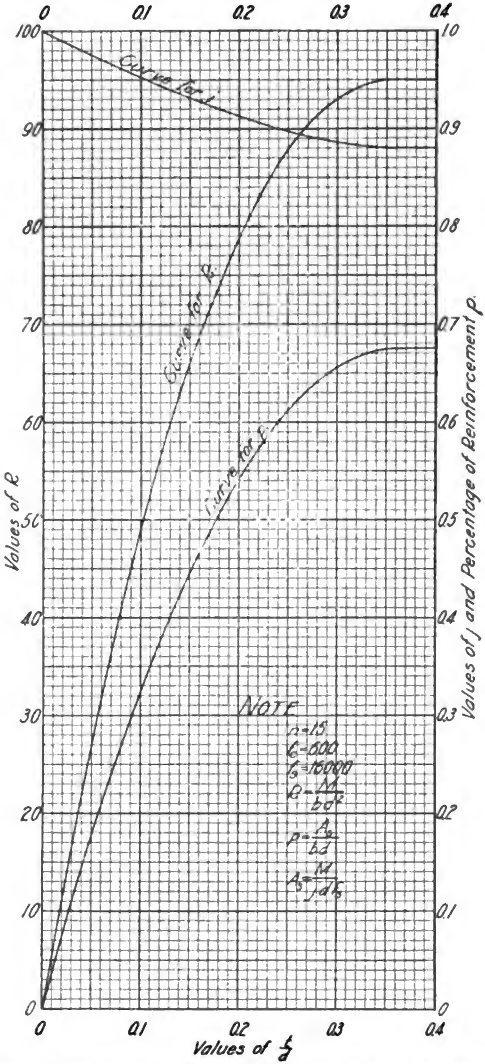


FIG. 37h′. Special Diagram for the Design of T-Beams when $f_c = 600$ and $f_s = 16{,}000$.

Finally, employing Equations 68 and 66, we have

$$b = \frac{10{,}000{,}000}{600\left(1 - \frac{5}{3} \times 0.2\right) \times 12 \times 60} = 35'',$$

and

$$A_s = \frac{10{,}000{,}000}{0.9 \times 60 \times 16{,}000} = 11.6 \text{ sq. in.}$$

2. What are the unit stresses in a T-beam subjected to a moment of 15,000,000 inch-pounds, on the assumption that $d = 60''$, $b = 50''$, $t = 12''$, and $p = 0.6\%$?

We first compute the values of $\frac{t}{d}$ and R, getting

$$\frac{t}{d} = \frac{12}{60} = 0.2,$$

and

$$R = \frac{15{,}000{,}000}{50 \times 60 \times 60} = 83.$$

We then enter the upper right-hand portion of Fig. 37*h* with $\frac{t}{d} = 0.2$ and $p = 0.6\%$, and trace vertically downward to the value $R = 83$, where we find the value of f_s to be about 15,200. Now entering the upper left-hand portion with the same values of $\frac{t}{d}$ and p, and tracing vertically downward to $f_s = 15{,}200$, we find $f_c = 620$.

Using now Equations 62 and 64, we have

$$A_s = 0.006 \times 60 \times 50 = 18.0 \text{ sq. in.}$$

$$f_s = \frac{15{,}000{,}000}{0.9 \times 60 \times 18} = 15{,}400,$$

and

$$f_c = \frac{15{,}000{,}000}{\left(1 - \frac{5}{3} \times 0.2\right) 50 \times 12 \times 60} = 620.$$

Case III. Neutral Axis in the Web; Compression in the Web Considered

Position of neutral axis,

$$kd = \sqrt{\frac{2ndA_s + (b-b')t^2}{b'} + \left[\frac{nA_s + (b-b')t}{b'}\right]^2} - \frac{nA_s + (b-b')t}{b'}. \quad \text{[Eq. 69]}$$

Position of resultant compression,

$$z = \frac{\left(kdt^2 - \frac{2}{3}t^3\right)b + \left\{(kd-t)^2\left[t + \frac{1}{3}(kd-t)\right]\right\}b'}{t(2kd-t)b + (kd-t)^2 b'}. \quad \text{[Eq. 70]}$$

Arm of resisting couple,

$$j\,d = d - z. \qquad \text{[Eq. 71]}$$

Coefficient of resistance,

$$R_s = f_s\,p\,j. \qquad \text{[Eq. 72]}$$

Moments of resistance,

$$M_s = f_s\,A_s\,(d - z) = f_s\,p\,j\,b\,d^2 = R_s\,b\,d^2, \qquad \text{[Eq. 73]}$$

$$M_c = \frac{f_c}{2\,k\,d}\left[(2\,k\,d - t)\,b\,t + (k\,d - t)^2\,b'\right](d - z). \qquad \text{[Eq. 74]}$$

Fibre stresses,

$$f_s = \frac{M}{A_s\;(d - z)} = \frac{M}{p\,j\,b\,d^2} \qquad \text{[Eq. 75]}$$

$$f_c = \frac{2\,M\,k\,d}{[(2\,k\,d - t)\,b\,t + (k\,d - t)^2\,b']\,(d - z)}. \qquad \text{[Eq. 76]}$$

Steel area,

$$A_s = \frac{M}{f_s\,(d - z)} = p\,b\,d. \qquad \text{[Eq. 77]}$$

The formulæ of Case III are so cumbersome that they are used but little. However, a T-beam can be considered to be made up of a rectangular portion of width b' and a T-beam portion of width $b - b'$ without any web, and the two portions figured separately and the results combined. Another scheme is to assume first that $b' = b$ and apply the formulæ and diagrams for rectangular beams, and second that $b' = 0$ and apply the formulæ for Case II; and then interpolate between these two figures to get the results for the actual value of $\frac{b'}{b}$. These methods are illustrated by the following problems:

1. Design the section of a T-beam for a moment of 10,000,000 inch-pounds, on the assumption that $d = 60''$, $t = 6''$, $b = 20''$, $f_c = 600$, and $f_s = 16{,}000$.

We first figure the moment of resistance of the rectangular portion, which is

$$M = 95 \times 20 \times 60^2 = 6{,}840{,}000 \text{ inch-pounds,}$$

the value of R being taken from Fig. 37*b* or 37*h*′, preferably the latter. The moment to be cared for by the T-beam portion is then

$$M = 10{,}000{,}000 - 6{,}840{,}000 = 3{,}160{,}000 \text{ in.-lbs.}$$

Entering now Fig. 37*h*′ with $\frac{t}{d} = \frac{6}{60} = 0.1$, we find $R = 49$; so that the value of $b - b'$ is

$$b - b' = \frac{3{,}160{,}000}{49 \times 60^2} = 18''.$$

The value of b is, therefore,

$$b = 18'' + 20'' = 38''.$$

The value of j for the rectangular portion is 0.88, and that for the T-beam portion is 0.95; so that the value of A_s is

$$A_s = \frac{6{,}840{,}000}{0.88 \times 60 \times 16{,}000} + \frac{3{,}160{,}000}{0.95 \times 60 \times 16{,}000}$$

$$= 8.1 \text{ sq. in.} + 3.5 \text{ sq. in.} = 11.6 \text{ sq. in.}$$

If we had disregarded the compression in the stem, we should have had

$$b = \frac{10{,}000{,}000}{49 \times 60^2} = 57'',$$

and $$A_s = \frac{10{,}000{,}000}{0.95 \times 60 \times 16{,}000} = 10.9 \text{ sq. in.}$$

This result indicates that it would have been improper in this case to neglect the compression in the stem.

The approximate formulæ for T-beams can also be used for this problem with small error. Employing them, we find

$$b - b' = \frac{3{,}160{,}000}{600\left(1 - \frac{5}{3} \times 0.1\right) 6 \times 60} = 18'',$$

$$b = 18'' + 20'' = 38'',$$

and $$A_s = 8.1 \text{ sq. in.} + \frac{3{,}160{,}000}{0.95 \times 60 \times 16{,}000} = 11.6 \text{ sq. in.}$$

If the compression in the stem were neglected, we should have

$$b = \frac{10{,}000{,}000}{600\left(1 - \frac{5}{3} \times 0.1\right) 6 \times 60} = 56'',$$

and $$A_s = \frac{10{,}000{,}000}{0.95 \times 60 \times 16{,}000} = 10.9 \text{ sq. in.}$$

2. Suppose that in Problem 2 under Case II the value of b' had been $20''$. Find the unit stresses, considering the compression in the stem.

We first find the value of $\frac{b'}{b}$, which is

$$\frac{b'}{b} = \frac{20}{50} = 0.4.$$

Neglecting the compression in the stem (which assumes that $\frac{b'}{b}$ is zero), we find, as before, that $f_c = 620$ and $f_s = 15{,}200$. Considering the stem to be 50″ wide $\left(\frac{b'}{b} = 1\right)$ and entering Fig. 37*b* with $R = 83$ and $p = 0.6\%$, we find $f_c = 550$ and $f_s = 15{,}600$. Interpolating for $\frac{b'}{b} = 0.4$, we get the results $f_c = 580$ and $f_s = 15{,}400$ The values of f_s and f_c with $\frac{b'}{b} = 1$ could also have been found by entering the upper portions of Fig. 37*h* at the intersections of the lines for $p = 0.6\%$ with the light dotted lines, which lines indicate the points at which k equals $\frac{t}{d}$.

RECTANGULAR BEAMS OF VARYING DEPTH

Position of neutral axis,

$$k = \sqrt{\frac{2\,p\,n\cos\beta'}{\cos^2\beta} + \frac{p^2 n^2 \cos^2\beta'}{\cos^4\beta}} - \frac{p\,n\cos\beta'}{\cos^2\beta} = \frac{n f_c}{f_s + n f_c}, \qquad \text{[Eq. 78]}$$

$$= \sqrt{2\,p_1 n + (p_1 n)^2} - p_1 n. \qquad \text{[Eq. 79]}$$

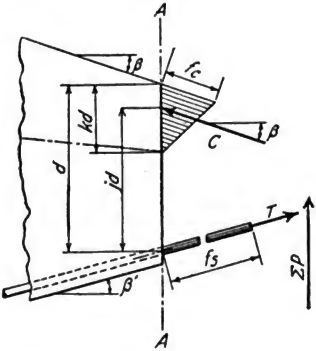


FIG. 37*i*.

Arm of resisting couple,

$$j = 1 - \tfrac{1}{3}k. \qquad \text{[Eq. 80]}$$

Coefficients of resistance,

$$R_s = f_s\,p\,j\cos\beta', \qquad \text{[Eq. 81]}$$

$$R_s' = f_s\,p_1\,j, \qquad \text{[Eq. 82]}$$

$$R_c = \tfrac{1}{2} f_c\,k\,j\cos^2\beta, \qquad \text{[Eq. 83]}$$

$$R_c' = \tfrac{1}{2} f_c\,k\,j. \qquad \text{[Eq. 84]}$$

Moments of resistance,

$$M_s = f_s\, p\, j\, b\, d^2 \cos\beta' = R_s\, b\, d^2, \qquad \text{[Eq. 85]}$$

$$M_c = \tfrac{1}{2} f_c\, k\, j\, b\, d^2 \cos^2\beta = R_c\, b\, d^2. \qquad \text{[Eq. 86]}$$

Fibre stresses,

$$f_s = \frac{M}{A_s\, j\, d \cos\beta'} = \frac{M}{p\, j\, b\, d^2 \cos\beta'} = \frac{f_c\, k \cos^2\beta}{2\, p \cos\beta'}, \qquad \text{[Eq. 87]}$$

$$f_c = \frac{2\, M}{k\, j\, b\, d^2 \cos^2\beta} = \frac{2\, f_s\, p \cos\beta'}{k \cos^2\beta}. \qquad \text{[Eq. 88]}$$

Steel ratio,

$$p = \frac{f_c\, k \cos^2\beta}{2 f_s \cos\beta'} = \frac{\cos^2\beta}{2\,\frac{f_s}{f_c}\left(\frac{f_s}{n\, f_c} + 1\right)\cos\beta'} = \frac{k^2 \cos^2\beta}{2\, n\,(1 - k)\cos\beta'}. \qquad \text{[Eq. 89]}$$

Cross-section of beam for given bending moment M,

$$b\, d^2 = \frac{M}{f_s\, p\, j \cos\beta'} = \frac{M}{R_s}, \qquad \text{[Eq. 90]}$$

$$= \frac{2\, M}{f_c\, k\, j \cos^2\beta} = \frac{M}{R_c}. \qquad \text{[Eq. 91]}$$

Fig. 37*j* applies for $n = 15$. The curves in the lower right-hand portion give simultaneous values of R'_s and R'_c (both called R'), p_1, f_c, and f_s; and the corresponding values of k and j can be found by projecting vertically upward to the scales for these quantities given at the top of the figure. The curves in the lower left-hand portion show simultaneous values of R, R', and β. The curves in the upper right-hand portion record simultaneous values of p_1, β, and $p \cos\beta'$; and the curves in the upper left-hand portion indicate simultaneous values of p, β', and $p \cos\beta'$. Ordinarily $\beta' = 0$, in which case $p \cos\beta' = p$, and we can read the values of p along the upper portion of the right-hand margin. When both β and β' are zero, $p_1 = p$ and $R' = R$, and we then need the lower right-hand diagram only. It will be noted that this diagram is identical with Fig. 37*b*, which latter figure applies only when β and β' are both zero.

The following problems illustrate the use of Fig. 37*j*:

1. A beam 18″ wide, having its tension face inclined at an angle of 20° and its compression face at angle of 30°, is to be designed for a moment of 4,000,000 inch-pounds, the allowable values of f_c and f_s being 600 and 16,000. What depth and reinforcement are necessary?

Entering the lower right-hand portion of Fig. 37*j* with $f_c = 600$ and $f_s = 16{,}000$, and tracing horizontally to $\beta = 30°$, we find $R = 71$; and on tracing vertically from the first point to the inclined line for $\beta = 30°$,

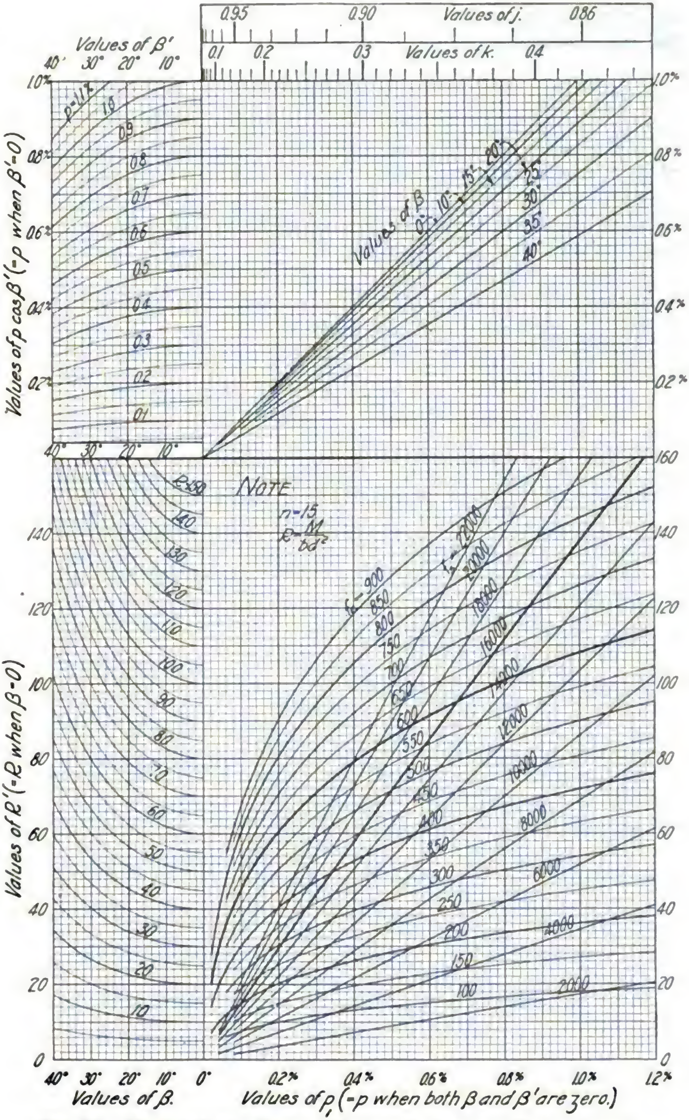


FIG. 37j. Diagram for the Design of Rectangular Beams of Varying Depth.

and then horizontally to $\beta' = 20°$, we find $p = 0.54\%$. We then compute the required depth, finding

$$d = \sqrt{\frac{4{,}000{,}000}{71 \times 18}} = 56''.$$

2. Suppose that in Problem 1 the value of β' had been zero instead of 20°. Design the section of the beam.

In this case we find $R = 71$ and $d = 56''$, as before; but to find the value of p we need merely to trace up to the inclined line for $\beta = 30°$, and then read directly the value of p as 0.51%.

3. Suppose that in Problem 1 the value of β had been zero. Design the section of the beam.

In this case we read the value of R directly at the intersection of the lines for $f_c = 600$ and $f_s = 16{,}000$, getting $R = 95$; and the required depth is then

$$d = \sqrt{\frac{4{,}000{,}000}{95 \times 18}} = 48''.$$

The value of p is found by tracing vertically upward to the inclined line for $\beta = 0$, and then horizontally to $\beta' = 20°$, which gives $p = 0.72\%$.

4. A beam for which $b = 12''$, $d = 40''$, and $p = 0.65\%$ is subjected to a moment of 1,500,000 inch-pounds. What are the unit stresses on the assumption that the tension face is inclined at 30°, and the compression face at 25°?

We first figure the value of R, finding

$$R = \frac{1{,}500{,}000}{12 \times 40 \times 40} = 78.$$

We then enter in the upper left-hand portion with $p = 0.65\%$ and $\beta' = 30°$, trace horizontally to the inclined line for $\beta = 25°$, and thence downward to a point found by projecting over horizontally from the intersection of the lines for $R = 78$ and $\beta = 25°$ in the lower left-hand portion. The values of f_c and f_s are then read as 600 and 16,000.

5. Suppose that in Problem 4 the value of β' had been zero. Compute the unit stresses.

As before, $R = 78$. We then enter the upper right-hand portion of Fig. 37j with $\beta = 25°$ and $p = 0.65\%$ (the horizontal lines representing the values of p in this case), and trace vertically downward to a point found by projecting over horizontally from the intersection of the lines for $R = 78$ and $\beta = 25°$. We then read the values of f_c and f_s as 570 and 13,800.

6. Suppose that in Problem 4 the value of β had been zero. What are the resulting unit stresses?

As in the other two cases, $R = 78$. We then enter the upper left-

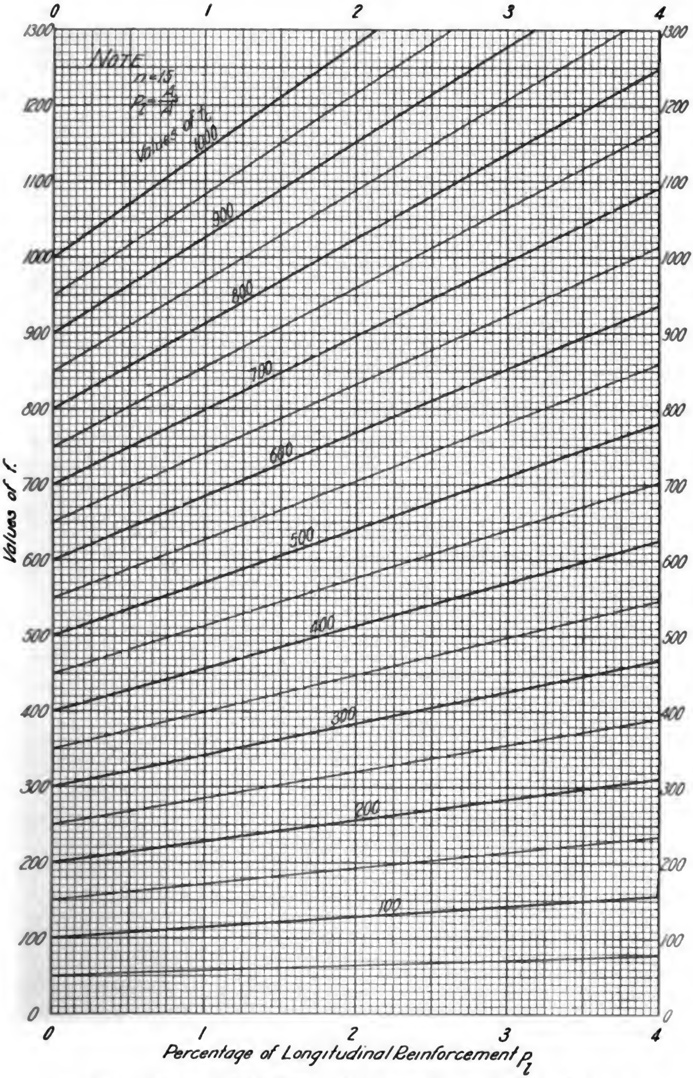


FIG. 37*k*. **Diagram for the Design of Columns Under Direct Stress Only.**

hand portion of Fig. 37*j* with $p = 0.65\%$ and $\beta' = 30°$, trace horizontally to the line $\beta = 0$, and then vertically downward to $R = 78$, given in this case by the horizontal lines. We then read $f_c = 520$ and $f_s = 15{,}600$.

COLUMNS UNDER DIRECT STRESS ONLY

Total safe direct load,

$$P = f_c\,(A_c + n\,A_l) = f_c\,A\,[1 + (n - 1)\,p_l] = f\,A. \qquad \text{[Eq. 92]}$$

Unit stresses,

$$f_c = \frac{P}{A\,[1 + (n - 1)\,p_l]} = \frac{f}{1 + (n - 1)\,p_l}, \qquad \text{[Eq. 93]}$$

$$f_s = n\,f_c, \qquad \text{[Eq. 94]}$$

$$f = \frac{P}{A} = f_c\,[1 + (n - 1)\,p_l]. \qquad \text{[Eq. 95]}$$

Fig. 37*k* is for $n = 15$. It gives simultaneous values of f, f_c, and p_l. No explanation of its use should be necessary.

RECTANGULAR BEAMS AND COLUMNS UNDER FLEXURE AND DIRECT STRESS, WITH REINFORCEMENT IN TENSION FACE ONLY. TENSION ON PART OF SECTION

Position of neutral axis,

$$\frac{k^2 - 2\,p\,n\,(1 - k)}{k^2\left(1 - \frac{k}{3}\right)} = \frac{d}{e'}. \qquad \text{[Eq. 96]}$$

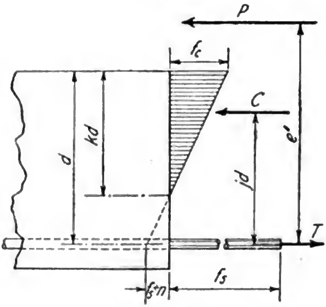


FIG. 37*l*.

Effective depth of beam,

$$j\,d = d\left(1 - \frac{k}{3}\right). \qquad \text{[Eq. 97]}$$

Coefficient of resistance,

$$R_c = \tfrac{1}{2} f_c \, k \, j. \qquad \text{[Eq. 98]}$$

Moment of resistance,

$$M_c = \tfrac{1}{2} f_c \, k \, j \, b \, d^2. \qquad \text{[Eq. 99]}$$

Fibre stresses,

$$f_s = n f_c \frac{1-k}{k}, \qquad \text{[Eq. 100]}$$

$$f_c = \frac{2\,M'}{j\,k\,b\,d^2} = \frac{2\,Pe'}{j\,k\,b\,d^2}, \qquad \text{[Eq. 101]}$$

Steel ratio,

$$p = \frac{k^2}{2\,n\,(1-k)} - \frac{k^2\left(1 - \frac{k}{3}\right)}{2\,n\,(1-k)} \cdot \frac{d}{e'}. \qquad \text{[Eq. 102]}$$

Fig. 37*m* applies for $n = 15$. The curves in the lower portion of the diagram give simultaneous values of R_c (or R), f_c, f_s, and k; and the corresponding value of j can be found by projecting vertically upward to the line marked "curve for j," and then tracing horizontally to the left-hand margin. The curves in the upper portion give simultaneous values of $\frac{d}{e'}$, p, and k. The full portions of the p curves are to be used when P is a compressive force and e' is positive, and the dotted portions when P is a tensile force and e' is negative. The negative values of $\frac{d}{e'}$ are given in parentheses along the left-hand margin.

The use of Fig. 37*m* is illustrated by the following problems:

1. The section at the bottom of the face wall of a retaining-wall of cantilever type is subjected to a moment from earth pressure of 40,000 foot pounds per lineal foot of wall, and a direct load of 6,000 pounds per lineal foot located one foot in front of the steel. Design the section on the assumption that $f_c = 600$ and $f_s = 16{,}000$.

We first figure the values of M' and e', which are

$$M' = 40{,}000 + 6{,}000 \times 1 = 46{,}000 \text{ ft.-lbs.},$$

and

$$e' = 46{,}000 \div 6{,}000 = 7.7'.$$

Entering Fig. 37*m* with $f_c = 600$ and $f_s = 16{,}000$, we find $R = 95$; so that the required value of d is

$$d = \sqrt{\frac{46{,}000 \times 12}{95 \times 12}} = 22''.$$

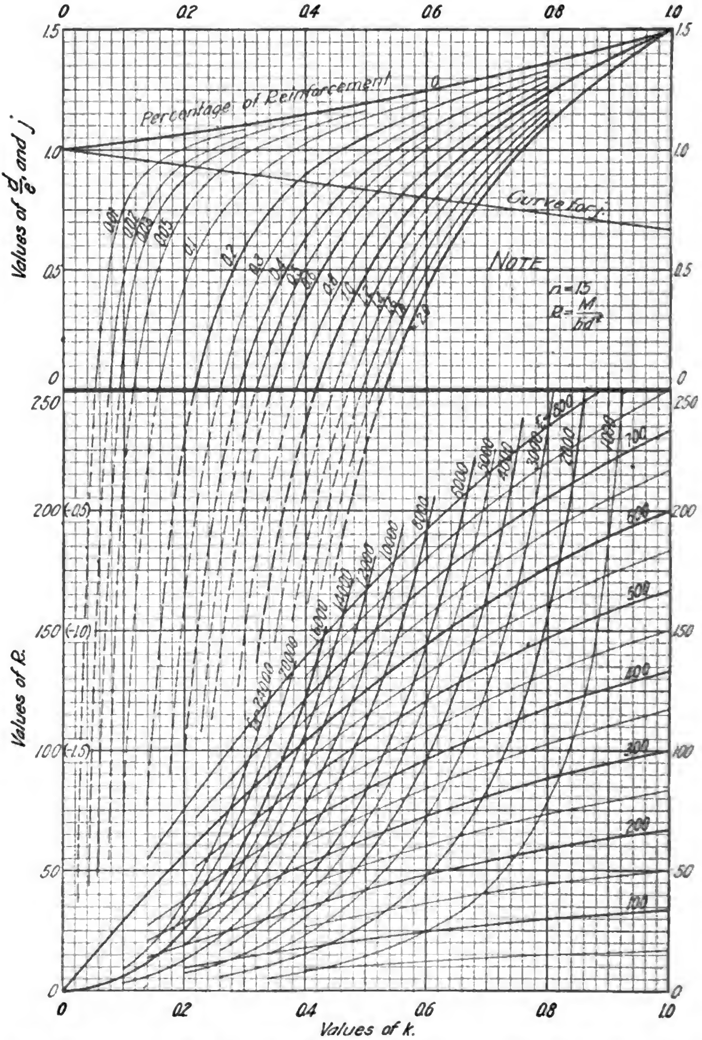


FIG. 37m. Diagram for the Design of Beams and Columns Under Flexure and Direct Stress, with Reinforcement in Tension Face Only.

We then compute the value of $\frac{d}{e'}$, finding

$$\frac{d}{e'} = \frac{22}{7.7 \times 12} = 0.24.$$

On tracing upward from the intersection of the lines for $f_c = 600$ and $f_s = 16{,}000$ on Fig. 37*m* to the point where $\frac{d}{e'} = 0.24$, we find that p equals 0.54%.

2. A pier shaft of an arch bridge is 8′ 0″ square, and is reinforced with 0.1% of steel placed 3″ from one face. It is subjected to a load of 1,800,000 pounds, located 2′ 6″ from the centre of the shaft and on the side opposite to the steel. What are the values of f_c and f_s?

We first compute the values of e', $\frac{d}{e'}$, and R, which are

$$e' = 30'' + 45'' = 75'',$$

$$\frac{d}{e'} = \frac{93}{75} = 1.24,$$

and

$$R = \frac{1{,}800{,}000 \times 75}{96 \times 93^2} = 163.$$

Entering the upper portion of Fig. 37*m* with $\frac{d}{e'} = 1.24$ and $p = 0.1\%$, and tracing vertically downward to the point where $R = 163$, we find $f_c = 650$ and $f_s = 5{,}500$. In order to find the amount of reinforcement which will reduce f_c to 600, we trace horizontally to the point where $R = 163$ and $f_c = 600$ and then trace vertically upward to $\frac{d}{e'} = 1.24$, and read the value of p as 0.3%.

RECTANGULAR BEAMS AND COLUMNS UNDER FLEXURE AND DIRECT STRESS, WITH REINFORCEMENT IN BOTH FACES

Case I. Compression on the Entire Cross-Section, Using the Transformed Section

Area, moment of inertia, and centroid of transformed section,

$$A_t = b\,h + n\,(A_s + A_s'), \qquad \text{[Eq. 103]}$$

$$I_t = I_c + n\,I_s, \qquad \text{[Eq. 104]}$$

$$c = \frac{\frac{1}{2}h + n\,p\,d + n\,p'\,d'}{1 + n\,p + n\,p'}. \qquad \text{[Eq. 105]}$$

Fibre stresses,

$$f_c = \frac{P}{A_t} + \frac{M\,c}{I_t}, \qquad \text{[Eq. 106]}$$

$$f_c' = \frac{P}{A_t} - \frac{M\,(h - c)}{I_t}, \qquad \text{[Eq. 107]}$$

$$f_s' = n\,\frac{P}{A_t} + \frac{n\,M\,(c - d')}{I_t}, \qquad \text{[Eq. 108]}$$

$$f_s = n\,\frac{P}{A_t} - \frac{n\,M\,(d - c)}{I_t}. \qquad \text{[Eq. 109]}$$

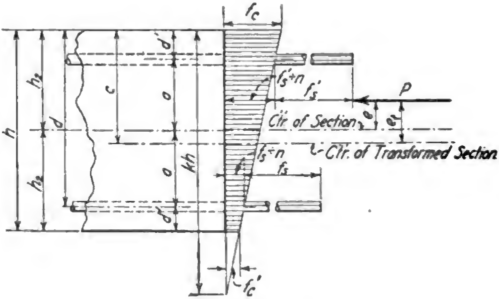


FIG. 37*n*.

If f_c' is negative, the problem comes under Case III.

Case II. Special Case of Case I in which the Reinforcement is Symmetrical, Using the Method Employed for Simple Flexure

(See Fig. 37*n*.)

Position of point where compressive stresses reduce to zero,

$$k = \frac{1 + 24\,n\,p\,\frac{a^2}{h^2} + 6\,(1 + 2\,n\,p)\,\frac{e}{h}}{12\,(1 + 2\,n\,p)\,\frac{e}{h}}. \qquad \text{[Eq. 110]}$$

Coefficient of resistance,

$$R_c = \frac{f_c}{12\,k}\left(1 + 24\,n\,p\,\frac{a^2}{h^2}\right). \qquad \text{[Eq. 111]}$$

Moment of resistance,

$$M_c = \frac{f_c\,b\,h^2}{12\,k}\left(1 + 24\,n\,p\,\frac{a^2}{h^2}\right) = R_c\,b\,h^2. \qquad \text{[Eq. 112]}$$

Fibre stresses,

$$f_c = \frac{12\,k\,M}{b\,h^2\left(1 + 24\,n\,p\frac{a^2}{h^2}\right)} = \frac{P}{bh}\left(\frac{1}{1 + 2\,n\,p} + 6\frac{e}{h}\cdot\frac{1}{1 + 24\,n\,p\frac{a^2}{h^2}}\right), \qquad \text{[Eq. 113]}$$

$$f_c' = f_c\left(1 - \frac{1}{k}\right) = \frac{P}{bh}\left(\frac{1}{1 + 2\,n\,p} - 6\,\frac{e}{h}\cdot\frac{1}{1 + 24\,n\,p\frac{a^2}{h^2}}\right), \qquad \text{[Eq. 114]}$$

$$f_s' = n\,f_c\left(1 - \frac{d'}{k\,h}\right), \qquad \text{[Eq. 115]}$$

$$= n\,f_c \text{ roughly}, \qquad \text{[Eq. 116]}$$

$$f_s = n\,f_c\left(1 - \frac{d}{k\,h}\right), \qquad \text{[Eq. 117]}$$

$$= n\,f_c' \text{ roughly}. \qquad \text{[Eq. 118]}$$

Fig. 37*o* is drawn for beams having symmetrical reinforcement, with n equal to 15 and $\frac{d'}{h}$ equal to 0.05, 0.1, and 0.15. It gives simultaneous values of p, $\frac{e}{h}$, $\frac{f_c}{f}$, and $\frac{f_c'}{f_c}$. It will be noted that the quantity $\frac{f_c}{f}$ is the ratio of f_c, the maximum concrete stress, to f or $\frac{P}{b\,h}$, the average stress on the entire section. The full curves only apply for Case II, the dotted curves being drawn in by means of the formulæ of Case III. Figure 37*o* thus handles all values of $\frac{e}{h}$ up to $\frac{1}{4}$, even though there be tension on part of the section.

The dotted curves in Fig. 37*q* apply for Case II. They are employed in the same manner as the full curves of that diagram, the use of which curves is explained under Case III. Owing to the large overlap of Figs. 37*o* and 37*q*, it will be unnecessary to compute $\frac{e}{h}$ accurately before deciding which diagram to employ. The figuring of the quantity $\frac{f_c}{f}$ or $\frac{f_c\,b\,h}{P}$ is a trifle simpler than that of the expression $\frac{M}{b\,h^2 f_c}$, so that ordinarily it will be best to use Fig. 37*o* unless $\frac{e}{h}$ exceeds $\frac{1}{4}$.

The following problems illustrate the use of Fig. 37*o*:

1. Design the reinforcement of an arch rib 3′ 4″ wide and 2′ 1″ deep, which is subjected to a direct stress of 450,000 pounds and a moment of 110,000 foot-pounds, for $f_c = 600$ and $f_s = 16{,}000$.

We first figure the value of e, getting

$$e = \frac{110{,}000}{450{,}000} = 0.25'.$$

Since $\frac{e}{h}$ is evidently about 1/8, Fig. 37*o* will be used. We therefore compute the value of $\frac{e}{h}$, which we find to be

$$\frac{e}{h} = \frac{0.25}{2.08} = 0.121.$$

Assuming the reinforcement to be placed 2½″ from each face, and to be symmetrical, the value of $\frac{d'}{h}$ becomes

$$\frac{d'}{h} = \frac{2.5}{25} = 0.1.$$

The value of f is

$$f = \frac{450{,}000}{25 \times 40} = 450;$$

and the allowable value of $\frac{f_c}{f}$ is, therefore,

$$\frac{f_c}{f} = \frac{600}{450} = 1.33.$$

The last two operations can be conveniently combined, giving

$$\frac{f_c}{f} = \frac{600 \times 25 \times 40}{450{,}000} = 1.33.$$

Entering now the central diagram of Fig. 37*o* with $\frac{f_c}{f} = 1.33$ and $\frac{e}{h} = 0.121$, we find the required value of p to be 0.73% per face. Since the value of $\frac{f_c'}{f_c}$ is about 0.25, f_c' equals 600 × 0.25, or 150.

2. Suppose that in Problem 1 the steel had been placed 2″ from the face of the concrete. Design the reinforcement.

In this case we have

$$\frac{d'}{h} = \frac{2}{25} = 0.08.$$

We therefore first enter the central portion of Fig. 37*o*, for which $\frac{d'}{h} = 0.1$, and find, as before, that p per face should be 0.73%. Entering now the lower portion, for which $\frac{d'}{h} = 0.05$, we get 0.65%. For $\frac{d'}{h} = 0.08$, we should evidently use about 0.70% per face.

3. An arch rib 4′ 0″ wide and 3′ 0″ deep, which is reinforced with 0.9% of steel in each face placed 2½″ from the surface of the con-

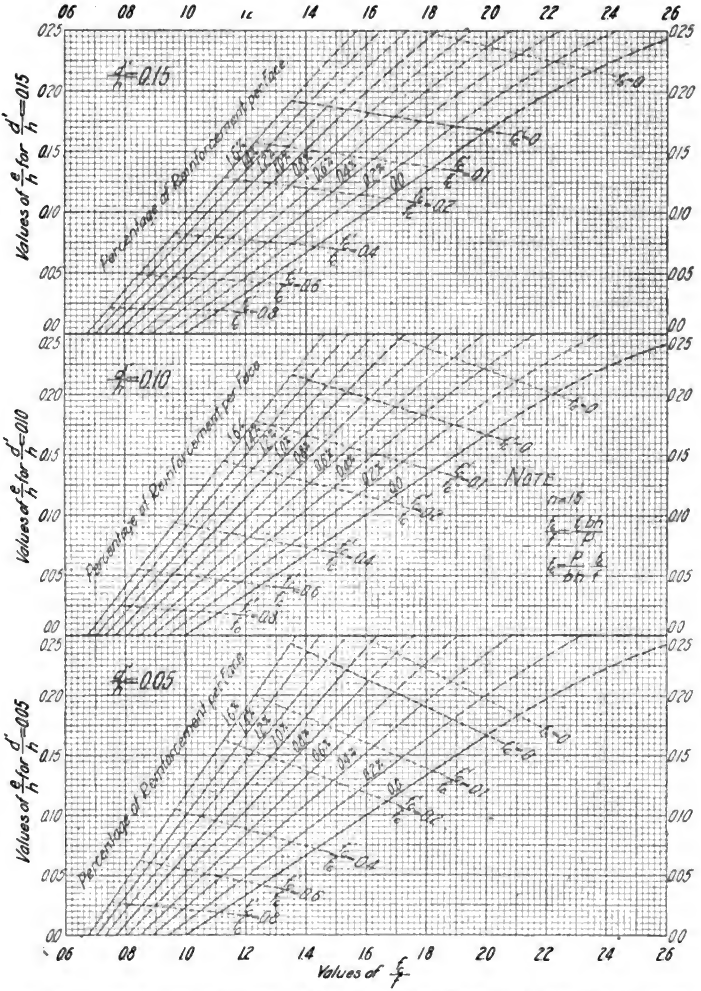


FIG. 37o. Diagram for the Design of Beams and Columns Under Flexure and Direct Stress, with Reinforcement in Both Faces—for $\frac{e}{h} < \frac{1}{4}$.

crete, is subjected to a direct load of 900,000 pounds and a bending moment of 180,000 foot-pounds. Compute the values of f_c and f_c'.

We first compute the values of $\frac{d'}{h}$ and e, getting

$$\frac{d'}{h} = \frac{2.5}{36} = 0.07,$$

and

$$e = \frac{180,000}{900,000} = 0.2'.$$

Since $\frac{e}{h}$ is evidently less than $\frac{1}{10}$, we must use Fig. 37*o*. We now compute the value of $\frac{e}{h}$, which is

$$\frac{e}{h} = \frac{0.2}{3} = 0.067.$$

Turning now to Fig. 37*o*, we first enter the lower diagram—which is for $\frac{d'}{h} = 0.05$—and find, for $\frac{e}{h} = 0.067$ and $p = 0.90\%$, that $\frac{f_c}{f}$ equals 1.03; and then enter the middle portion, for which $\frac{d'}{h} = 0.10$, and get for the value of $\frac{f_c}{f}$ 1.05. For $\frac{d'}{h} = 0.07$, the value of $\frac{f_c}{f}$ will, therefore, be 1.04; and f_c will then be

$$f_c = \frac{900,000}{36 \times 48} \times 1.04 = 540.$$

The value of $\frac{f_c'}{f_c}$ is about 0.55, so that $f_c' = 0.55 \times 540 = 300$.

4. Suppose that the arch rib in Problem 3 had been subjected to a direct load of 600,000 pounds and a bending moment of 400,000 foot-pounds. Compute the extreme fibre stresses.

Figuring first the values of $\frac{d'}{h}$ and e, we have

$$\frac{d'}{h} = \frac{2.5}{36} = 0.07,$$

and

$$e = \frac{400,000}{600,000} = 0.67'.$$

Since $\frac{e}{h}$ is evidently a trifle less than $\frac{1}{4}$, we can employ Fig. 37*o*. We then calculate the value of $\frac{e}{h}$, which is

$$\frac{e}{h} = \frac{0.67}{3} = 0.22.$$

Turning now to Fig. 37*o*, and entering with $\frac{e}{h} = 0.22$ and $p = 0.9\%$, we find the value of $\frac{f_c}{f}$ to be 1.58 when $\frac{d'}{h} = 0.05$, and 1.64 when $\frac{d'}{h} = 0.10$; so that for $\frac{d'}{h} = 0.07$, $\frac{f_c}{f}$ equals 1.6. The value of f_c is, therefore,

$$f_c = \frac{600{,}000}{36 \times 48} \times 1.6 = 560.$$

Since the dotted curves are used, there is tension over a part of the section; and the value of f_s is practically zero.

Case III. Tension on Part of the Cross-Section—Special Case in Which the Reinforcement is Symmetrical

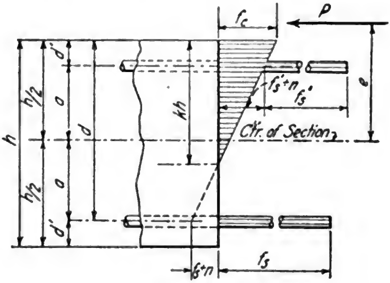


FIG. 37*p*.

Position of neutral axis,

$$k^3 - 3\left(\frac{1}{2} - \frac{e}{h}\right) k^2 + 12\, n\, p \frac{e}{h} k = 6\, n\, p \left(\frac{e}{h} + 2 \frac{a^2}{h^2}\right). \qquad \text{[Eq. 119]}$$

Coefficient of resistance,

$$R_c = f_c \left[\frac{k}{12}(3 - 2\,k) + \frac{2\,p\,n}{k} \frac{a^2}{h^2}\right]. \qquad \text{[Eq. 120]}$$

Moment of resistance,

$$M_c = f_c\, b\, h^2 \left[\frac{k}{12}(3 - 2\,k) + \frac{2\,p\,n}{k} \frac{a^2}{h^2}\right] = R_c\, b\, h^2. \qquad \text{[Eq. 121]}$$

Fibre stresses,

$$f_c = \frac{M}{b\, h^2 \left[\frac{k}{12}(3 - 2\,k) + \frac{2\,p\,n}{k} \frac{a^2}{h^2}\right]}, \qquad \text{[Eq. 122]}$$

$$f_s' = n\, f_c \left(1 - \frac{d'}{k\,h}\right), \qquad \text{[Eq. 123]}$$

$$f_s = n f_c \left(\frac{d}{k\,h} - 1\right) = n f_c \frac{(1-k)\,h - d'}{k\,h}. \qquad \text{[Eq. 124]}$$

Fig. 37*q* is drawn for beams having symmetrical reinforcement, with n equal to 15 and $\frac{d'}{h}$ equal to 0.05, 0.1, and 0.15. It gives simultaneous values of $p, \frac{h}{e}, \frac{R}{f_c}$, and $\frac{f_s}{f_c}$ for cases in which there is tension on part of the section, and of $p, \frac{h}{e}, \frac{R}{f_c}$, and $\frac{f'_c}{f_c}$ for cases in which there is compression over the entire section. The full curves apply in the first instance, and the dotted ones in the second. The dotted curves cover a range of values of $\frac{h}{e}$ which is also taken care of by Fig. 37*o*. For values of $\frac{e}{h}$ ranging from $\frac{1}{7}$ to $\frac{1}{4}\left(\frac{h}{e}\text{ from 7 to 4}\right)$, either of the two figures can be used for any plotted value of p; and Fig. 37*q* applies down to $\frac{e}{h} = \frac{1}{10}, \left(\frac{h}{e} = 10\right)$, for values of p exceeding 0.7%. As stated under Case II, the calculations required with Fig. 37*o* are a trifle simpler than those with Fig. 37*q*, so that Fig. 37*o* should ordinarily be employed unless $\frac{e}{h}$ exceeds $\frac{1}{4}\left(\text{or } \frac{h}{e}\right.$ is less than $\left.4\right)$.

The following problems illustrate the use of Fig. 37*q*:

1. Design the reinforcement of an arch rib 3′ 4″ wide and 2′ 1″ deep, which is subjected to a direct stress of 300,000 pounds and a moment of 240,000 foot-pounds, for $f_c = 600$ and $f_s = 16{,}000$.

We first figure the value of e, getting

$$e = \frac{240{,}000}{300{,}000} = 0.8'.$$

Since $\frac{e}{h}$ is evidently over ⅓, Fig. 37*q* must be employed. We therefore compute the value of $\frac{h}{e}$, which we find to be

$$\frac{h}{e} = \frac{2.08}{0.8} = 2.6.$$

Assuming the reinforcement to be symmetrical and located 2½″ from each face, the value of $\frac{d'}{h}$ becomes

$$\frac{d'}{h} = \frac{2.5}{25} = 0.1.$$

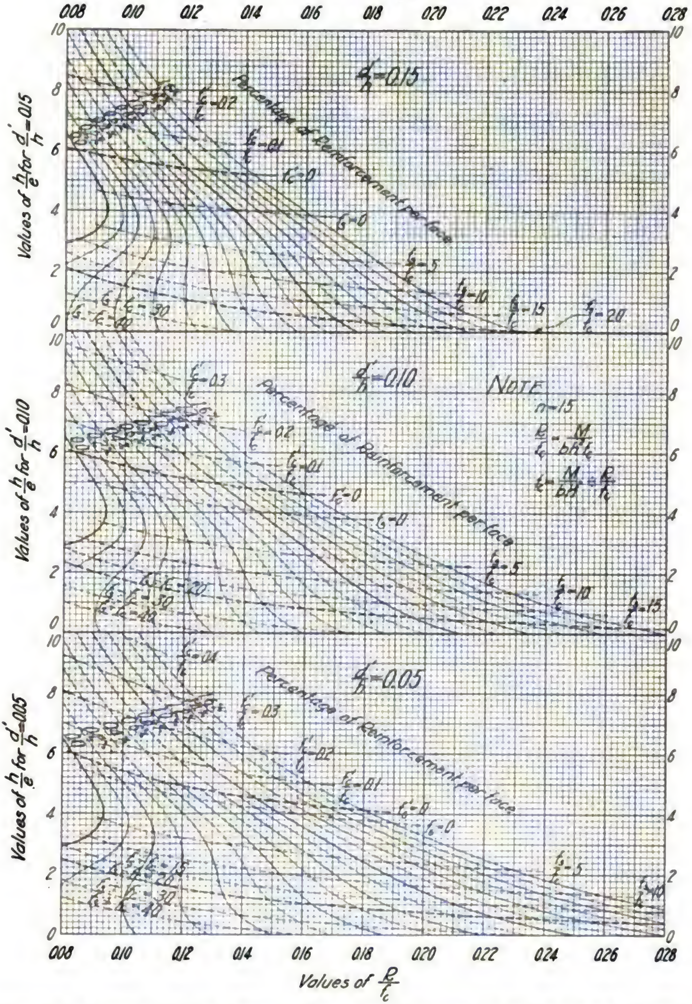


FIG. 37*q*. Diagram for the Design of Beams and Columns under Flexure and Direct Stress, with Reinforcement in Both Faces—for $\frac{e}{h} > \frac{1}{7}$.

We next figure the value of $\frac{R}{f_c}$ or $\frac{M}{b\,h^2 f_c}$, which is

$$\frac{R}{f_c} = \frac{240{,}000 \times 12}{40 \times 25^2 \times 600} = 0.192.$$

Entering the central portion of Fig. 37q with $\frac{h}{e} = 2.6$ and $\frac{R}{f_c} = 0.192$, we find the required value of p to be 1.5% per face. The value of $\frac{f_s}{f_c}$ at this point is about 5, so that f_s equals 5 × 600, or 3,000.

2. Suppose that in Problem 1 the steel had been placed 2″ from the face of the concrete. Design the reinforcement.

In this case we have

$$\frac{d'}{h} = \frac{2}{25} = 0.08.$$

We therefore first enter the central portion of Fig. 37q, for which $\frac{d'}{h} = 0.1$, and find, as before, that p per face should be 1.5%. Entering now the lower portion, for which $\frac{d'}{h} = 0.05$, we get 1.22%. For $\frac{d'}{h} = 0.08$, we should evidently use 1.4% per face. The value of f_s is about 5 × 600, or 3,000.

3. An arch rib 4′ 0″ wide and 3′ 0″ deep, which is reinforced with 1.2% of reinforcement in each face placed 2½″ from the surface of the concrete, is subjected to a direct load of 750,000 pounds and a bending moment of 360,000 foot-pounds. Compute the value of f_c, and of f_s or f'_c.

We first figure the values of $\frac{d'}{h}$ and e, which are

$$\frac{d'}{h} = \frac{2.5}{36} = 0.07,$$

and

$$e = \frac{360{,}000}{750{,}000} = 0.48.$$

Since $\frac{e}{h}$ is about ⅙, Fig. 37o would be the best one to use; but Fig. 37q will be employed in order to illustrate the application of the dotted curves thereon. We next compute the value of $\frac{h}{e}$, which we find to be

$$\frac{h}{e} = \frac{3}{0.48} = 6.25.$$

Entering first the lower part of Fig. 37q with $p = 1.2\%$ and $\frac{h}{e} = 6.25$, we find, for $\frac{d'}{h} = 0.05$, that $\frac{R}{f_c}$ equals 0.128; and on entering the central portion, for which $\frac{d'}{h} = 0.10$, we find that $\frac{R}{f_c}$ equals 0.123. For $\frac{d'}{h} = 0.07$, $\frac{R}{f_c}$ will therefore be 0.126. The value of f_c is then found to be

$$f_c = \frac{360{,}000 \times 12}{48 \times 36^2 \times 0.126} = 550.$$

Since the dotted lines were used, there is compression over the entire section; and the value of f'_c is, approximately,

$$f_c = 550 \times 0.15 = 80.$$

Had Fig. 37*o* been used, we would first have figured the value of $\frac{e}{h}$, which is

$$\frac{e}{h} = \frac{0.48}{3} = 0.16.$$

We then find, entering Fig. 37*o* with $\frac{e}{h} = 0.16$ and $p = 1.2\%$, that $\frac{f_c}{f}$ equals 1.25 when $\frac{d'}{h}$ is 0.05, and 1.30 when $\frac{d'}{h}$ is 0.10; so that its value, when $\frac{d'}{h} = 0.07$, becomes 1.27. The value of f_c is, therefore,

$$f_c = \frac{750{,}000}{48 \times 36} \times 1.27 = 550;$$

and the value of f'_c is, approximately,

$$f'_c = 550 \times 0.15 = 80.$$

Shear, Bond, and Web Reinforcement

Rectangular Beams of Constant Depth

Maximum unit shear on cross-section,

$$v = \frac{V}{j\,b\,d} = \frac{u\,\Sigma\,o}{b}. \qquad \text{[Eq. 125]}$$

Unit bond stress,

$$u = \frac{V}{j\,d\,\Sigma\,o} = \frac{v\,b}{\Sigma\,o}. \qquad \text{[Eq. 126]}$$

Amount of shear carried by concrete,

$$V_c = v_c\,j\,b\,d \qquad \text{[Eq. 127]}$$

Amount of shear taken by web reinforcement,

$$V_w = V_i + V_v = V - V_c = V - v_c\,j\,b\,d. \qquad \text{[Eq. 128]}$$

Portion of unit shear v taken by the web reinforcement,

$$v_w = v_i + v_v = v - v_c. \qquad \text{[Eq. 129]}$$

Portion of unit shear v taken by bars inclined at 45°,

$$v_i = \frac{1.4\, m_i\, A_i\, f_s}{s_i\, b} = \frac{1.4\, m_i\, P_i}{s_i\, b} = \frac{V_i}{b\, j\, d}. \qquad \text{[Eq. 130]}$$

Amount of shear carried by bars inclined at 45°,

$$V_i = b\, j\, d\, v_i = \frac{1.4\, m_i\, A_i\, f_s\, j\, d}{s_i} = \frac{1.4\, m_i\, P_i\, j\, d}{s_i}. \qquad \text{[Eq. 131]}$$

Stress in one bar inclined at 45°,

$$P_i = \frac{0.7\, v_i\, s_i\, b}{m_i} = \frac{0.7\, V_i\, s_i}{j\, d\, m_i} = A_i f_s. \qquad \text{[Eq. 132]}$$

Unit stress in bars inclined at 45°,

$$f_s = \frac{P_i}{A_i} = \frac{0.7\, v_i\, s_i\, b}{m_i\, A_i} = \frac{0.7\, V_i\, s_i}{j\, d\, m_i A_i}. \qquad \text{[Eq. 133]}$$

Required spacing of vertical stirrups,

$$s_v = \frac{m_v\, A_v\, f_s}{v_v\, b} = \frac{m_v\, A_v\, f_s\, j\, d}{V_v} = \frac{m_v\, P_v}{v_v\, b} = \frac{m_v\, P_v\, j\, d}{V_v}. \qquad \text{[Eq. 134]}$$

Unit shear carried by vertical stirrups,

$$v_v = \frac{m_v\, A_v\, f_s}{s_v\, b} = \frac{m_v\, P_v}{s_v\, b} = \frac{V_v}{b\, j\, d}. \qquad \text{[Eq. 135]}$$

Amount of shear carried by vertical stirrups,

$$V_v = b\, j\, d\, v_v = \frac{m_v\, A_s\, f_s\, j\, d}{s_v} = \frac{m_v\, P_v\, j\, d}{s_v}. \qquad \text{[Eq. 136]}$$

Stress in one bar of a vertical stirrup,

$$P_v = \frac{v_v\, s_v\, b}{m_v} = \frac{V_v\, s_v}{j\, d\, m_v} = A_v f_s. \qquad \text{[Eq. 137]}$$

Unit stress in vertical stirrups,

$$f_s = \frac{P_v}{A_v} = \frac{v_v\, s_v\, b}{m_v\, A_v} = \frac{V_v\, s_v}{j\, d\, m_v\, A_v}. \qquad \text{[Eq. 138]}$$

The value of j may be taken as ⅞ with sufficient accuracy in all calculations for shear and bond stresses.

Fig. 37*r* can be used for the design of vertical stirrups composed of ⅜″, ½″, and ¾″ round bars, and for the design of web reinforcement consisting of ½″, ¾″, ⅞″, 1″, and 1¼″ round bars inclined at 45°. For the ⅜″ and ½″ bars, the diagrams record simultaneous values of v_v, m per foot width of beam, and s; and in the case of the ½″ bars a scale for values of v_i is also given at the right, although it will rarely be needed. For the other bars, the diagram gives simultaneous values of v_i, m per foot width, and s; and in the case of the ¾″ bars a scale for the values

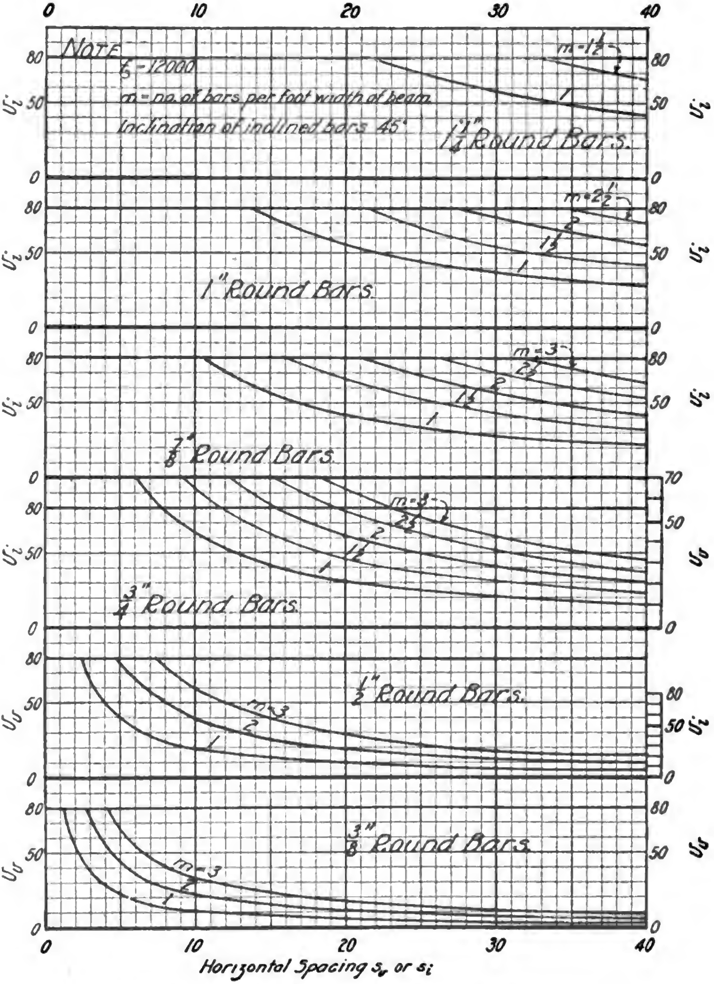


Fig. 37r. Diagram for the Design of Web Reinforcement.

of v_v is given at the right, although it will hardly ever be used. Values for ⅝″ round bars can be found by interpolating between the results obtained for ½″ and ¾″ bars, and those 1⅛″ round bars, by interpolating between the results found for 1″ and 1¼″ bars. The use of Fig. 37*r* is illustrated on page 932 *et seq*.

T-Beams of Constant Depth

Equations 125 to 138 apply unchanged, except that b is to be replaced by b'. The quantity $j\,d$ may be taken as $d - \frac{t}{2}$ when $\frac{t}{d} < 0.25$, and as $\frac{7}{8}\,d$ when $\frac{t}{d} > 0.25$, with sufficient accuracy in all cases. Fig. 37*r* can be employed.

Beams of Varying Depth

Amount of shear to be carried by concrete and web reinforcement,

$$V_1 = V_c + V_w = V - j\,(C \sin \beta + T \sin \beta')$$

$$= V - j\,T \cos \beta' \,(\tan \beta + \tan \beta') = V - \frac{M}{d}(\tan \beta + \tan \beta'). \quad \text{[Eq. 139]}$$

(β and β' are to be taken as positive when they bear the relation to the direction of ΣP shown in Fig. 37*i*, and negative when they are the reverse.)

Equations 125 to 138, inclusive, apply unchanged, except that V is to be replaced by V_1; and Fig. 37*r* can be employed.

THE POSITIONS OF CRITICAL SECTIONS FOR PURE SHEAR AND DIAGONAL TENSION IN BEAMS

The determination of the positions of the critical sections for pure shear is not difficult. They are to be taken at the edges of supports, or at the edges of the load areas of concentrated loads. The location of the critical section for diagonal tension in a beam, however, is somewhat uncertain. The results of available tests will first be discussed briefly, and then working rules will be given.

A considerable number of tests have been made on simply supported beams carrying concentrated loads at the third points. An extensive series of such tests was reported by Prof. A. N. Talbot in Bulletin No. 29 of the Engineering Experiment Station of the University of Illinois. In the case of the beams without web reinforcement which failed by diagonal tension, the failure crack was almost invariably a diagonal one, located about midway between the load point and the support. In the case of similar beams having web reinforcement, a diagonal crack usually formed at about the same location and load as in the beam without web reinforcement; and then at a load somewhat greater a similar crack frequently formed at the supports. Since the shear was constant from load point to support, these results indicate that the amount of diagonal tension developed near the supports or the load points was less than that which occurred at intermediate points. This was doubtless due to the fact that at these intermediate points true beam action prevailed, while near the supports and the load points there was a considerable amount of arch action.

If this latter explanation is the true one, we should expect the shearing strength of a beam loaded in the manner just discussed to be increased by moving the loads nearer to the supports. On page 166 of Turneaure and Maurer's "Principles of Reinforced Concrete Construction" there is described a series of tests on beams simply supported and loaded with two equal concentrated loads symmetrically placed, the locations of these loads being varied from the centre of the beam to points near the supports. These beams were of 4-foot span, and were 4½ inches deep. The unit shearing strengths developed by the beams for various positions of the loads are given in the third column of the following table:

Distance from Load Point to Support		Unit Shearing Stress Lbs./ins.²	Assumed Unit Diagonal Tensile Stress Lbs./ins.²	Ratio of Assumed Unit Diagonal Tensile Stress to Unit Shearing Stress Lbs./ins.²
In inches	In Terms of Depth of Beam *d*			
24	5.3 *d*	177	177	1.0
18	4.0 *d*	200	177	0.89
12	2.7 *d*	220	177	0.81
8	1.8 *d*	316	177	0.56
6	1.3 *d*	512	177	0.35
4	0.9 *d*	850	177	0.21
2	0.5 *d*	1035	177	0.17

The rapid increase as the loads approach the supports is apparent. If we assume the unit diagonal tensile stress developed to be 177 pounds per square inch in all cases, the ratios of the unit diagonal tensile stresses to the unit shearing stresses will have the values given in the fifth column of the table. These values should be considered very rough ones only, until they are checked by other tests; but it is evident that loads applied to the top of a beam near the supports will produce much smaller diagonal tensile stresses than those which act at a greater distance from the ends of the beam. Similar conclusions can be drawn from some tests of simply supported beams under uniform loading, described on page 151 *et seq.* of Prof. Mörsch's "Der Eisenbetonbau." In each of these beams failure occurred by the formation of a diagonal crack at a distance from the supports about equal to the depth of beam, although the unit shear at this point was considerably less than that at the edge of the support.

In Bulletin No. 67 of the Engineering Experiment Station of the University of Illinois will be found the results of tests on a number of wall footings. In such of these footings as failed in diagonal tension, the failure crack started at the reinforcement at a distance from the face of the wall about equal to the depth of the footing, and extended diagonally upward to the junction of the face of the wall and the top of the footing. The critical section for diagonal tension seems, therefore, to have been located at a distance from the face of the wall equal to the depth of the footing. The nominal vertical shearing stresses at this critical section were about twice those usually found in beam tests, and the values at the face of the wall were about forty per cent larger yet. The length of the footing projection was 2.4 times the depth in most instances. From the ratios given in the fifth column of Fig. 37*a* we should expect the diagonal tension at the critical section, due to loads placed from 2.4*d* to 1.0*d* from the edge of the support, to be about half of the nominal vertical shearing stress at the same point, which value agrees with the results given in the bulletin.

The same bulletin also gives the results of some tests on square column footings. The location of the critical section appears to have been about the same as for the wall footings, but the nominal unit shearing stress at this section was much lower. This latter effect is doubtless due to the fact that the section for resisting diagonal tension is less than that for resisting the nominal vertical shear, since the four planes along which the diagonal tension failure occur taper toward the pier. Also, it is possible that the shear was not distributed uniformly over the critical section. The nominal unit shear on the section at the face of the pier was found to be about the same as for the wall footings.

In all tests of beams which have come to the author's notice, the loading has consisted of compressive forces acting on one face thereof, rather than tensile forces applied to one face, as in the case of a through girder. It is quite certain that the resistance to diagonal tension would be low with this last-mentioned kind of loading, since it induces direct tensile stresses in addition to the diagonal tension.

The various tests which have been cited show that the manner of loading of a beam may influence decidedly the amount of diagonal tension developed therein and also the position of the critical section. Unfortunately, they are not sufficiently extensive to warrant the drawing of definite conclusions. The author suggests the following working rules, based on the tests and upon a study of the stress trajectories of beams of various kinds. The rules undoubtedly err considerably on the side of safety; but in the present state of our knowledge this condition is unavoidable.

For a slab or beam carrying a heavy, moving, concentrated load, the edge of the load is to be placed at a distance equal to the depth *d* of the said slab or beam from the edge of the support, and the unit diagonal tensile stress on the portion of the beam between the load and the edge of the support is to be considered equal to the unit vertical shearing stress thereon.

For a wall footing, the critical section is to be considered located at a distance from the face of the wall equal to the depth *d*, and the unit diagonal tensile stress on this section is to be assumed equal to the unit vertical shearing stress thereon.

For a column footing, the critical section is to be considered located at a distance equal to half the depth *d* from the face of the pier, and the unit diagonal tensile stress on this section is to be assumed equal to the unit vertical shearing stress thereon.

For a beam which carries a uniform loading, the critical section for diagonal tension near a support on which the beam is simply supported is to be taken at the edge of the said support; while near a support over which the beam is continuous, it is to be taken at a distance from the edge of the support equal to half the depth of the beam. In both cases the unit diagonal tensile stress is to be assumed equal to the unit shearing stress on the critical section.

For beams on which the load is concentrated at several load points, or on which the load is distributed but not uniform, the rules just given for beams carrying uniform loading are to be used.

Moments of Inertia of Beams, Columns, and Arch Ribs

The moment of inertia of a reinforced concrete beam is a rather uncertain quantity, due to variations in the modulus of elasticity of the concrete and to the effect of the cracking thereof on the tension side of the beam. At low unit stresses in the steel—say, 3,000 pounds per square inch or less—the concrete will still exert nearly its full strength in tension; but at ordinary working stresses a large portion of its tensile resistance will have disappeared. The 1913 Report of the Joint Committee suggests that n be taken as eight ($E_c = 3{,}750{,}000$), and that the tensile strength of the concrete be considered zero. The accuracy of this assumption can be tested by checking the deflections of beams figured on this assumption against the results obtained in actual tests. In Bulletins 28 and 29 of the Engineering Experiment Station of the University of Illinois, and in Bulletin No. 197 of the University of Wisconsin, there will be found the deflection curves of a large number of rectangular beams and T-beams. If the deflection curves for these beams be computed on the assumption just given, and then plotted on the diagrams, it will be found to give very satisfactory results when the steel stresses are about 16,000 pounds per square inch. In view of this fact, the author considers it to be the best rule to follow. It must not be forgotten, however, that it is only approximate, and that in any particular case the true value may differ considerably from the one obtained in this manner.

The moment of inertia of a rectangular reinforced concrete beam may be expressed by the formula,

$$I = \alpha b d^3. \qquad \text{[Eq. 140]}$$

The value of α, when the tensile strength of the concrete is ignored, is given by the equation,

$$\alpha = \tfrac{1}{3}\left[k^3 + 3\,n\,p\,(1 - k)^2\right], \qquad \text{[Eq. 141]}$$

the value of k being that given by Equation 9. The lower curve in Fig. 37s gives values of α for various values of p and for $n = 8$. For comparison, the upper curve has been added to give the value of α when the full tensile resistance of the concrete is considered. For this latter case the equation for α takes the form,

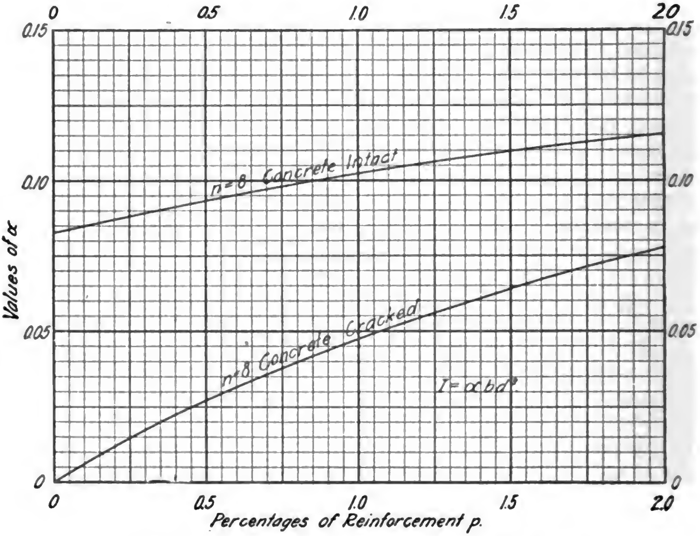


FIG. 37s. Moments of Inertia of Rectangular Beams.

$$\alpha = \tfrac{1}{3}\,[k^3 + (1 - k)^3 + 3\,n\,p\,(1 - k^2)], \qquad \text{[Eq. 142]}$$

the value of k being given by the equation,

$$k = \frac{1 + 2\,n\,p}{2 + 2\,n\,p}. \qquad \text{[Eq. 143]}$$

The development of the last two equations can be found on page 120, *et seq.*, of Turneaure and Maurer's "Principles of Reinforced Concrete Construction."

For values of p less than 0.5 per cent, the lower curve will undoubtedly give results somewhat too low.

It might be noted that if n be taken as fifteen ($E_c = 2{,}000{,}000$), and the moment of inertia of the concrete section only be used, the relative value of α will be $\frac{1}{12} \times \frac{8}{15}$, or 0.044. This value is seen to agree fairly well with the lower curve for ordinary percentages of steel; and where there is no great variation in the percentage of reinforcement in the various portions of a girder, the use of this approximate rule is entirely satisfactory.

The moment of inertia of a T-beam may be found with sufficient accuracy in the same manner as for a rectangular beam. For values of $\frac{t}{d}$ much less than those of k, a lower value should, theoretically, be obtained; but as the width of flange to be considered is generally indeterminate, any attempt at refinement is unjustified. The width of the flange should usually be taken somewhat larger than that used in designing the section.

At the supports of continuous T-beams some allowance must be made for the tensile strength of the concrete in the slab, which increases the stiffness of the beam materially. A much smaller width should be taken than that assumed at the centre of the span, partly because the slab will be cracked at intervals, and partly because the point of contraflexure in the beam is so close to the support that the stress cannot spread very far transversely. The simplest manner of figuring the moment of inertia of such a section is to compute the area of the slab for the assumed width and divide it by 8, and then consider the area of the reinforcement to be increased by this amount.

In case a beam which carries a slab is designed as a rectangular beam, T-beam action must be considered in figuring the moment of inertia.

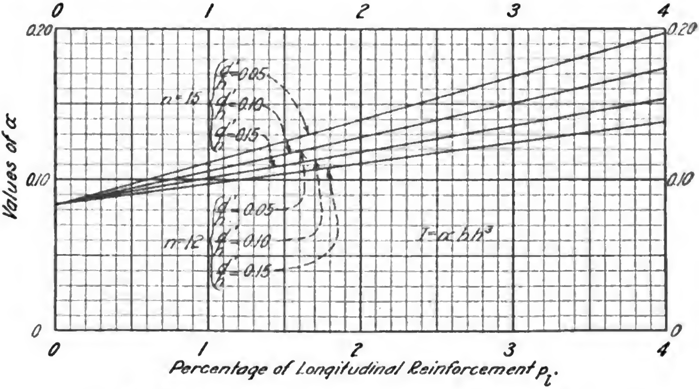


FIG. 37*l*. Moments of Inertia of Columns and Arch Ribs.

The moment of inertia of a column depends almost entirely upon the value of n, as there will rarely be any extensive cracking of a properly designed column. It is best to take the value of n as twelve (E_c = 2,500,000), since that is its most probable value at ordinary working stresses.

A good discussion concerning the choice of the value of n for columns is to be found on page 243, *et seq.*, of Turneaure and Maurer's "Principles of Reinforced Concrete Construction."

The moment of inertia of an arch rib should be figured for the entire section, although it is likely to be cracked at the points where the maximum stresses occur. To allow for the effect of such cracking, and also for the fact that the permissible unit stresses are higher than in the case of a column, n should be taken as fifteen ($E_c = 2{,}000{,}000$).

Fig. 37*t* gives values of α for columns and arch ribs with symmetrical reinforcement, for $n = 12$ and $n = 15$, and for various values of p_l and $\frac{d'}{h}$. It is assumed that all of the reinforcement is in the two faces of the column or rib, the percentage p_l representing the sum of the steel areas in the said two faces.

THE CALCULATION OF THE DEFLECTIONS OF BEAMS AND ARCHES

The deflections of reinforced concrete beams should be figured by the ordinary formulæ for homogeneous beams, the moments of inertia being computed as explained in the preceding section of this chapter. The resulting values must not be considered as in any way exact; but they will indicate roughly the deflections that may be expected.

The deflections of arch ribs should be computed by the method given on page 179 of Part II of "Modern Framed Structures," by Johnson, Bryan, and Turneaure (1910 edition), or on page 344 of Turneaure and Maurer's "Principles of Reinforced Concrete Construction." If $\frac{ds}{I}$ is not assumed constant (see the section of this chapter entitled "The Calculation of Stresses in Arch Ribs with Fixed Ends"), it must be put under the summation sign. The solution can also be made graphically, as is explained in Chapter XII on page 238.

THE CALCULATION OF STRESSES IN MONOLITHIC STRUCTURES

In the design of monolithic reinforced concrete structures it is necessary to take into account the continuity of the various portions. The method of calculation must, of course, be based on the elastic properties of the different members. The resulting stresses will be but approximately correct, as no investigation of a continuous structure can take into account all of the factors involved. This is particularly true in the case of reinforced concrete construction, because of the fact that the stiffness of a reinforced concrete beam decreases in an uncertain manner as the unit stresses therein increase, due to the progressive cracking of the concrete. Approximate methods of calculation are generally to be considered satisfactory. It is essential, however, that the nature of the stresses at all sections be correctly determined, even though no attempt be made to get exact values thereof, and that reinforcement be provided at all points where tension is likely to exist; for otherwise unsightly, or even dangerous, cracks are likely to occur.

Chapter XI treats of the calculation of deflections of beams; and Chapter XII gives methods to be used for computing the stresses in continuous beams in general. The principles therein presented can be applied to reinforced concrete structures. The Theorem of Three Moments will be found useful when not more than two stiff members meet at any point, and the Theorem of Four Moments when there are more than two such members so meeting. Equations 1 to 42, inclusive, of Chapter XI can frequently be employed, especially for approximate investigations; and Equations 43 and 44 of the same chapter can be used to analyze the stresses in a continuous structure of any kind. The methods presented in Bulletin 80 of the Engineering Experiment Station of the University of Illinois will also be found valuable.

In Turneaure and Maurer's "Principles of Reinforced Concrete Construction" there is given a complete analysis of the moments in continuous beams of two and three equal spans, for uniform loads only; and somewhat similar information is to be found on page 435 of "Concrete, Plain and Reinforced," by Taylor and Thompson. A much fuller discussion of the subject of stresses in continuous members is to be found in Part II of Hool's "Reinforced Concrete Construction." This latter book gives tables and diagrams for determining moments and shears in beams of two and three equal spans for both uniform and concentrated loads. It also treats of the calculation of stresses in columns continuous with girders. All three of the books just mentioned discuss the selection of approximate coefficients for use in ordinary design work. The 1913 Report of the Joint Committee also recommends values for such coefficients.

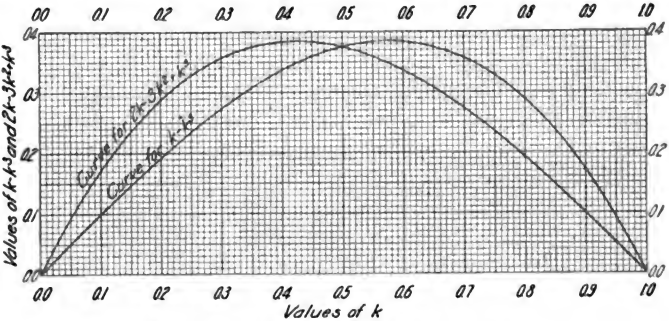


FIG. 37u. Values of $k - k^3$ and $2k - 3k^2 + k^3$.

Fig. 37u will be found valuable when making exact calculations for the stresses produced by concentrated loads. It gives values of $k - k^3$

and $2k - 3k^2 + k^3$, k having its usual significance in continuous girder formulæ.

Fig. 37*v* can be used for figuring the moments (produced by uniform loads) at the supports of continuous beams in which the span lengths are unequal and the moment of inertia is constant throughout the length of each span, but not necessarily the same in the different spans. It applies directly to three-span girders, but can be extended to serve for other cases. The diagram gives values of the coefficients A, B, and C in the equation,

$$M_1 = -A\,w_1\,l_1^2 - B\,w_2\,l_2^2 + C\,w_3\,l_3^2, \qquad \text{[Eq. 144]}$$

in which the various symbols have the meanings indicated in the sketch of the beam shown thereon. The values of A, B, and C were determined by the Theorem of Three Moments.

In using this diagram, it is necessary to enter at the lower margin with the value of c_1 $\left(\text{or } \frac{l_1/I_1}{l_2/I_2}\right)$, and trace vertically to the curves for the value of c_3 $\left(\text{or } \frac{l_3/I_3}{l_2/I_2}\right)$, reading the values of the coefficients A, B, and C at the right-hand margin. By analogy, coefficients for the value of M_2 can evidently be found by the formula,

$$M_2 = C\,w_1\,l_1^2 - B\,w_2\,l_2^2 - A w_3\,l_3^2, \qquad \text{[Eq. 145]}$$

using $c_1 = \frac{l_3/I_3}{l_2/I_2}$ and $c_2 = \frac{l_1/I_1}{l_2/I_2}$.

When the moment of inertia is constant throughout the three spans, the above ratios evidently become $\frac{l_1}{l_2}$ and $\frac{l_3}{l_2}$.

To illustrate the use of the diagram, suppose that we wish to find the values of M_1 and M_2 in a three-span continuous girder in which l_1, l_2, and l_3 are 20 feet, 40 feet, and 30 feet, respectively, and the moment of inertia is constant throughout the entire girder. We then proceed as follows:

Moment M_1.

$$c_1 = \frac{20}{40} = 0.5,$$

$$c_3 = \frac{30}{40} = 0.75.$$

$$\begin{aligned} \therefore M_1 &= -0.046\,w_1 \times 20^2 - 0.066\,w_2 \times 40^2 + 0.020\,w_3 \times 30^2, \\ &= -18.4\,w_1 - 106\,w_2 + 18\,w_3. \end{aligned}$$

Moment M_2.

$$c_1 = \frac{30}{40} = 0.75,$$

$$c_3 = \frac{20}{40} = 0.5.$$

$$\therefore M_2 = 0.013\, w_1 \times 20^2 - 0.52\, w_2 \times 40^2 - 0.059\, w_3 \times 30^2,$$
$$= 5.2\, w_1 - 83\, w_2 - 53\, w_3.$$

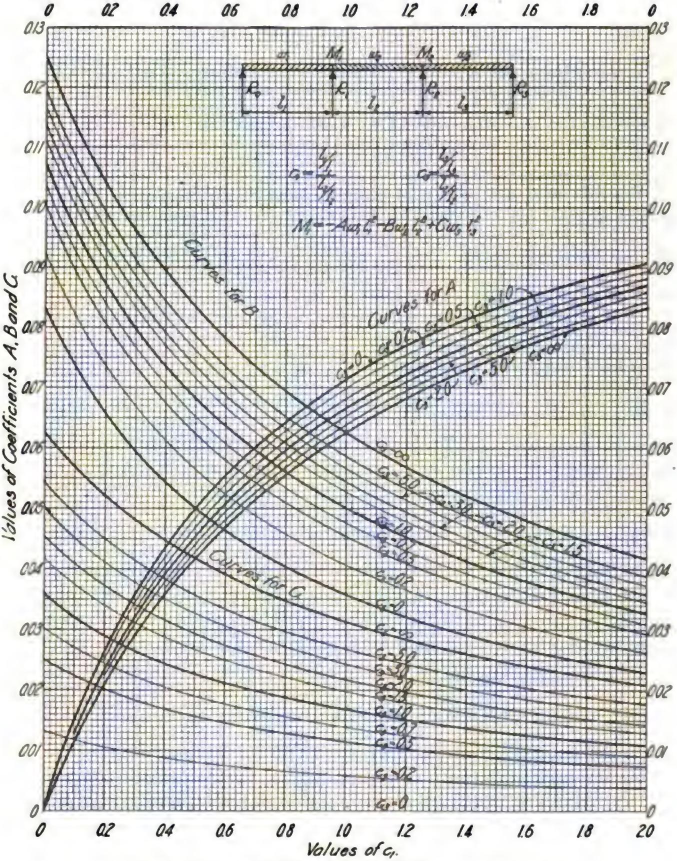


FIG. 37v. Moment Coefficients for Continuous Beams.

This diagram can be applied to a two-span girder simply supported at the ends by letting l_1 and l_2 represent the two spans, and putting $\frac{l_3}{I_3} = \infty$, (assuming $l_3 = \infty$ or $I_3 = 0$), under which circumstances the moment at R_2 due to loads on spans l_1 and l_2 is zero. For instance, if we had such a girder in which the spans were 20 feet and 40 feet and the moment of inertia constant throughout, we could apply the diagram in the following manner:

$$\begin{aligned} \text{Let } l_1 &= 20, \\ l_2 &= 40, \\ \text{and } l_3 &= \infty. \\ \therefore c_1 &= 0.5, \\ \text{and } c_3 &= \infty. \\ \therefore M_1 &= 0.042\,w_1 \times 20^2 + 0.083\,w_2 \times 40^2, \\ &= 16.8\,w_1 + 133\,w_2. \end{aligned}$$

Fig. 37*v* also will serve for a two-span continuous girder simply supported at one end and fixed at the other. In this case we proceed as in the last instance, except that $\frac{l_3}{I_3}$, and therefore c_3, is taken as zero, which assumption makes the beam fixed at R_2.

If in any given case the span l_3 should be fixed at the support R_3, the moments M_1 and M_2, due to loads w_1 and w_2, will be the same as though the span l_2 were only three-fourths (¾) as long and were simply supported at R_3; and the moment at R_3 will be equal to $-\frac{M_2}{2}$. If the same span be continuous over R_3 with a span of equal stiffness—*i.e.*, of equal length and moment of inertia—the equivalent span length for a span simply supported at R_3 will be about ⅞ l_3; and the moment at R_3 will be approximately equal to $-\frac{M_2}{4}$. Should the adjacent span be stiffer, the equivalent length ratio will vary toward ¾ as a limit, and the moment ratio toward − ½; while if it be more flexible, the length ratio will vary toward unity, and the moment ratio toward zero. The truth of the above statements regarding the equivalent span lengths can be determined from Equations 7, 14, and 21 of Chapter XI; for from Equations 7 and 21 we find the ratio $\frac{3}{4}$, and from Equations 14 and 21 the ratio $\frac{3}{3.46}$, or $\frac{7}{8}$. The moment ratios are given by Equations 10 and 12 of the same chapter.

By the application of the principles given in the preceding paragraph, Fig. 37*v* can be used for practically any continuous girder; and they will be found very valuable in many other instances.

Suppose, for example, that we wish to find the moments at the supports of the four-span continuous girder shown in Fig. 37*w*, the moment of inertia of the girder being constant throughout. For loads w_1 and w_2, we can consider the length of l_3 to be ⅞ × 30, or 26.3; and for loads w_3 and w_4, we can assume the length of l_2 as ⅞ × 40, or 35. We then have:

Loads w_1 and w_2.

For M_2.

$$c_1 = \frac{20}{40} = 0.5, \text{ and } c_3 = \frac{26.3}{40} = 0.66.$$

$$\therefore M_2 = -0.046 \times 20^2 \times w_1 - 0.065 \times 40^2 \times w_2,$$
$$= -18.4\,w_1 - 104.0\,w_2.$$

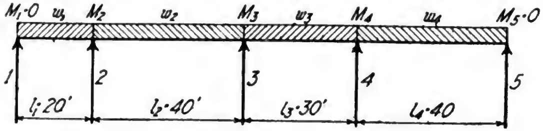


FIG. 37*w*. Sketch of a Four-Span Continuous Girder.

For M_3.

$$c_1 = 0.66, \text{ and } c_3 = 0.5.$$

$$\therefore M_3 = +0.014 \times 20^2 \times w_1 - 0.055 \times 40^2 \times w_2,$$
$$= 5.6\,w_1 - 88.0\,w_2.$$

For M_4.

$$M_4 = -1.4\,w_1 + 22.0\,w_2.$$

Loads w_3 and w_4.

For M_4.

$$c_1 = \frac{40}{30} = 1.33, \text{ and } c_3 = \frac{35}{30} = 1.17.$$

$$\therefore M_4 = -0.075 \times 40^2 \times w_4 - 0.044 \times 30^2 \times w_3,$$
$$= -120.0\,w_4 - 39.6\,w_3,$$

For M_3.

$$c_1 = 1.17, \text{ and } c_3 = 1.33.$$

$$\therefore M_3 = +0.017 \times 40^2 \times w_4 - 0.047 \times 30^2 \times w_3,$$
$$= 27.2\,w_4 - 42.3\,w_3.$$

For M_2.

$$M_2 = -6.8\,w_4 + 10.6\,w_3.$$

Collecting, we have

$$M_2 = -18.4\,w_1 - 104.0\,w_2 + 10.6\,w_3 - 6.8\,w_4,$$
$$M_3 = +5.6\,w_1 - 88.0\,w_2 - 42.3\,w_3 + 27.2\,w_4,$$
and
$$M_4 = -1.4\,w_1 + 22.0\,w_2 - 39.6\,w_3 - 120.0\,w_4.$$

The diagram just given assumes, as do all other methods in common use, that the moment of inertia of each span is constant throughout its length. The variations from this condition which ordinarily occur can be neglected without appreciable error. The question is discussed in Hool's "Reinforced Concrete Construction," Part II, page 390, in Turneaure and Maurer's "Principles of Reinforced Concrete Construction," page 320, and in Taylor and Thompson's "Concrete, Plain and Reinforced," page 430. If in any case it should be necessary to take into account variations in the moments of inertia, the moment-area method of calculating deflections, as explained in Chapter XII, can be employed to advantage, especially if it is necessary to use graphical methods of integration. An example of the calculation of stresses in columns with variable moments of inertia is to be found in a paper by E. E. Howard, Esq., M. Am. Soc. C. E., in the *Proceedings* Am. Soc. C. E. for May, 1915, entitled "The Twelfth Street Trafficway Viaduct." The computations were the work of E. A. Slettum, Esq., C. E., and were made for the author's firm, which designed the structure and supervised its construction.

The values of the moment at any point of a beam can be easily determined as soon as the moments at the supports have been figured, as can also the shears at the ends. For a beam which is continuous over one support and simply supported at the other, the maximum shear at the continuous end is about $^5/_8$ of the load on the span, and at the free end about $^7/_{16}$. If a beam is continuous over both supports, the dead load shears at each end will be about half of the load on the span, while the maximum live load shear at either end will be about $^9/_{16}$ of the said load. These ratios hold when the various spans are nearly alike. The dead load shear at any point within a span can be computed very easily. The maximum live load shear at any point can be figured with sufficient accuracy by calculating its value with the span fully loaded, and then figuring the effect of removing the load between the point and the support as though the beam were simply supported.

The method of computing the moments of inertia of beams and columns has already been explained in the section of the chapter entitled "Moments of Inertia of Beams, Columns, and Arch Ribs."

In all cases some allowance must be made for the uncertainties in the assumptions on which the solution is based, and for the possible settlement of the supports. If the effective lengths are taken as the distances from centre to centre of supports or as the clear span lengths plus twice the depth of the beam or slab, they will be enough greater than the true effective lengths to give moments some ten per cent in excess of the true values in most instances—which is sufficient to provide for ordinary conditions.

For the design of continuous slabs, standard coefficients should be used. For uniform loads, the moments should be taken equal to $\frac{w\,l^2}{12}$ at all supports and at the centres of all spans, where w is the load per lineal foot and l the effective span length, except for two-span slabs. The above value is a trifle low for the support next to the end one and for the centre portion of an end span; and on this account the value $\frac{w l^2}{10}$ is used by some designers. Since the supports are of considerable width, the former value is safe enough; and it is the one recommended by the Joint Committee. For two-span slabs the moments at the centre support and at the centre of each span should be taken as $\frac{w l^2}{10}$. For a single concentrated load P, the moment near the centre of an end span is about $\frac{P l}{5}$, and of an intermediate span about $\frac{P l}{6}$. For similar loads P at the centres of two adjacent spans, the moment at the intervening support will be a trifle less than $\frac{P l}{5}$. When l is taken as the distance from centre to centre of supports, the use of the formula $\frac{P l}{6}$ for the moments at all supports and at the centres of all spans is justified.

For small beams under uniform loading, except in the case of beams continuous for two spans only, the moments at the centres of intermediate spans and over intermediate supports should be taken as $\frac{w l^2}{12}$, and those near the centres of the end spans and over the adjacent support as $\frac{w l^2}{10}$. For single concentrated loads P placed at the centres of the spans, the corresponding values should be $\frac{P l}{6}$ and $\frac{P l}{5}$. For beams continuous over two spans only and carrying uniform loading, the moment at the centre of each span should be taken as $\frac{w l^2}{10}$, and over the middle support, as $\frac{w l^2}{8}$; while for single concentrated loads P placed at the centres of the spans, the formula $\frac{P l}{5}$ can be used for the moments at the centres of the spans and at the centre support.

In the design of large continuous girders in which the various spans are of equal length, the data given in Turneaure and Maurer's "Principles of Reinforced Concrete Construction," Taylor and Thompson's "Concrete, Plain and Reinforced," and Part II of Hool's "Reinforced Concrete Construction," and previously referred to, will be found to afford the easiest solution. When the span lengths are quite different and the loading is uniform, Fig. 37*v* should be used. Even when a portion of the loading is concentrated, it will usually be sufficiently accurate to assume it as uniform when the number of concentrations is three or more, and when the total load per foot is constant. The Theorem of Three Moments can be used if an exact analysis is to be made for concentrated loads. If the moment of inertia of a span varies, a special solution will be required to give exact results. Quite frequently the moment of inertia at the supports is greater than at the centre. In this case fairly accurate results can be obtained by figuring the moments on the assumption that the moment of inertia is constant, and then increasing the values at the supports somewhat, and likewise decreasing those at the centres of the spans.

The moment at the centre of an intermediate span, when figured by continuous girder formulæ, will frequently be found quite small. In order to ensure that proper sections will be used for such spans, a minimum moment coefficient should be adopted. This should not in any case be less than $\frac{w\,l^2}{20}$; and unless the foundations are excellent and the designer is to supervise the construction, a somewhat larger value, such as $\frac{w\,l^2}{16}$ or $\frac{w\,l^2}{12}$, should be used.

In figuring stresses in continuous girders, the stiffness of the supports is usually neglected. This leads to very small errors when the various spans are of equal length. The negative moment at the end of a span which is longer than an adjacent span will be increased by the stiffness of the support, and the positive moment at its centre diminished; while in a span which is shorter than the adjacent ones a reverse effect will occur. A fairly accurate value of the moment produced in a column can be found by figuring approximately the angle through which the axis of the girder has rotated at the top of the column, assuming the said top of column to rotate through the same angle, and then applying Equations 43 and 44 of Chapter XI. The value thus found will be on the safe side, since the stiffness of the column will reduce somewhat the angle through which the top thereof rotates. The effect of the stiffness of the columns on the stresses in the girders can be figured by assuming the moment at the top of each column to be divided between the two girders it carries in proportion to their ratios of I over l. As a rough working-rule, it may be assumed that the bending moments produced in a column by either the cross girders or the main girders will add about 100 pounds per square inch to the unit stresses produced by the direct loads only on the concrete at the section just below the girders.

If the top of a column is moved horizontally by the contraction or expansion of girders resting thereon, the resulting stresses in the column can be figured directly by means of Equations 39 to 42, inclusive, of Chapter XI; but Equations 43 and 44 of that chapter will also apply. If the column is battered, it will be sufficiently accurate to use the width at a point one-quarter of the distance below the top in figuring the unit stress at the top, and the width at a point one-quarter of the distance above the bottom in figuring the unit stress at the bottom.

The moments produced in the columns of a bent by loads on the cross girder can be computed by means of Fig. 37v; for in order that this diagram be applicable it is not necessary that the various spans of the girder lie in one straight line. It is essential, however, that there be no movement of the tops of the columns in any direction. If the columns are fixed at the bottom, only ¾ of their lengths should be used in computing the values of c_1 and c_3. If they are battered, the moment of inertia should be taken somewhat less than the average in figuring the moment at the upper end—say that at a point ⅓ or ¼ of the distance below the top; and if the columns are fixed at the base, the moment at the bottom will be somewhat greater than half of that at the top.

The effect of eccentric loads on columns can be figured by means of Equation 30 of Chapter XI.

The stresses produced in columns and girders by wind and traction loads can be figured by means of Equations 43 and 44 of Chapter XI. The application of these equations is well illustrated in Bulletin 80 of the Engineering Experiment Station of the University of Illinois.

THE CALCULATION OF STRESSES IN RECTANGULAR SLABS REINFORCED IN TWO DIRECTIONS

When a rectangular slab reinforced in two directions is loaded with a uniform load over its entire area, or with a concentrated load at its central point, the loading will divide between the two systems of reinforcement approximately in proportion to the relative stiffness of the slab in the two directions. The stiffness of a beam subjected to a uniform load is inversely proportional to the fourth power of its length, and that of one carrying a single concentrated load at the centre to the cube of its length. If a slab, the dimensions of which are l_1 and l_2, carries a uniform load of w per unit area, the portion w_1 thereof carried by the reinforcement parallel to the length l_1 is approximately that given by the formula,

$$w_1 = w\frac{l_2^4}{l_1^4 + l_2^4}. \qquad \text{[Eq. 146]}$$

Similarly, if the same slab carry a load P at its central point, the portion P_1 carried by the reinforcement parallel to l_1 can be found by the formula,

$$P_1 = P\frac{l_2^3}{l_1^3 + l_2^3}. \qquad \text{[Eq. 147]}$$

Equations 146 and 147 assume that the slab has the same degree of fixity over the supports in the two directions, since the stiffness of a beam is, to a large extent, dependent upon the condition of the ends. A beam fixed at both ends is about four or five times as stiff as one free at both ends, and a beam continuous over both supports is about twice as stiff as one free at both ends. This factor should be taken into account when there is a great difference in the amount of fixity in the two directions; but with most slabs it should be ignored, especially as Equations 146 and

147 are at best but approximate, and, consequently, refinement is not justified.

If the length of the slab in one direction exceeds that in the other direction by more than fifty per cent, it will be best to assume the entire load to be carried by the reinforcement running in the shorter direction.

The moments per foot of width in the two directions should be figured by means of the coefficients given in the preceding section of this chapter, making due allowance for the fixity of the ends. A thorough discussion of the moments in the slab itself and of the distribution of the loads on the supporting beams is to be found on page 308 of Turneaure and Maurer's "Principles of Reinforced Concrete Construction." Uniform loads only are considered, but the method of analysis can be applied to concentrated loads as well.

If a slab carries a load which is spread over a considerable area at the centre, rather than concentrated at a point, it will be best to calculate the distribution in the two directions as though it were actually concentrated at a point. Should the loaded area be considerably longer in one direction than the other (as in the case of the front wheel of a road-roller), it will still be sufficiently accurate to follow this same procedure. The dimensions of the load should be used, however, in determining the width over which it is to be distributed, and also in computing the moments.

The maximum shears from a concentrated load will be nearly the same as for a slab reinforced in one direction only, for the reason that when such a load is placed near a support, practically the entire amount will be carried by one system of reinforcement. When a load is spread over a considerable area, the above statement may not be true.

THE DISTRIBUTION OF CONCENTRATED LOADS OVER SLABS

The question of the distribution of concentrated loads over slabs is an important one to the bridge engineer, since the roadway floor-slab in nearly every bridge is to be designed for either a road-roller or a motor-truck; and it is very desirable to know the width of a slab which will be effective for carrying such loads. Unfortunately, there is but little experimental knowledge bearing on the subject. A few tests were reported in a paper by A. T. Goldbeck, Esq., C.E., which was published in the 1913 Proceedings of the American Society for Testing Materials. These tests indicated that the effective width was about eight-tenths (0.8) of the span length. In a discussion of Mr. Goldbeck's paper, W. A. Slater, Esq., C.E., gave the results of some tests made at the University of Illinois. Mr. Slater suggested that when a concentrated load is placed at the centre of a slab supported on two sides only, the resulting moment should be considered to distribute over an effective width b_m given by the formula,

$$b_m = l \tan \theta, \qquad \text{[Eq. 148]}$$

in which l is the span of the slab, and tan θ depends upon the ratio of the width of the slab to its span length. He gave a curve for the values of tan θ. A fairly extensive series of tests is reported in Bulletin No. 28 of the Highway Department of the State of Ohio, which was issued in September, 1915. This report suggests that the effective width be taken as six-tenths (0.6) of the span length plus one and seven-tenths (1.7) feet. This rule is based upon the conclusion that the effective width, in percentage of the span length, decreases as the span length increases. Such a deduction, however, appears to be unwarranted; for while the tests seem at first glance to be in agreement therewith, this apparent agreement is due to the fact that the applied load was distributed over a considerable width, rather than concentrated at a point. If the widths of the load areas be subtracted from the effective widths given in the Bulletin, and the remainders be divided by the corresponding span lengths, the values of the resulting ratios will be found to be affected but slightly by the differences in the span lengths. The effective widths determined in this manner indicate values somewhat higher than those given by Mr. Slater's curve.

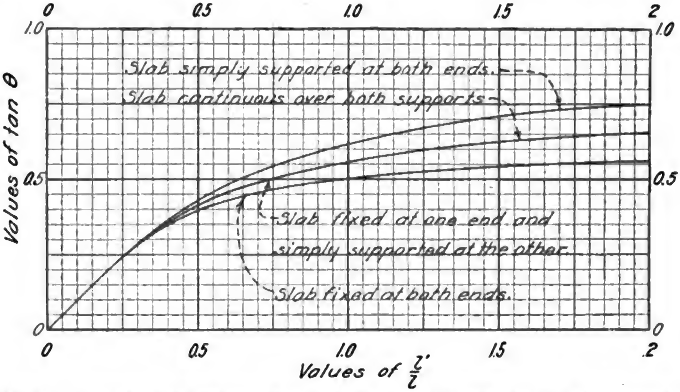


FIG. 37x. Diagram for Determining the Distribution of Concentrated Loads over Slabs.

Until further experiments have been performed, it would appear advisable to use the formula suggested by Mr. Slater. The upper curve in Fig. 37x is of the same form as Mr. Slater's curve, but it has been drawn in a trifle higher to agree with the results of the Ohio tests. It gives values of tan θ for use in Equation 148, for various ratios of the width of the slab l' to its span length l.

Equation 148 applies directly only to a single load concentrated at a point at the centre of a slab which is supported along two sides only and is not continuous over these supports. It is impossible to determine with much accuracy the effect of variations from this ideal condition; but the following rough working rules will handle practical problems in as satisfactory a manner as is possible at the present time.

If a load is distributed over a considerable area rather than concentrated at a point, the following method can be employed. In Fig. 37y suppose *ABCD* to represent a slab simply supported along the lines *AB* and *CD*, and unsupported along *AD* and *BC*. Suppose a load *P* to be uniformly distributed over an area of length *c* and width *w*, the centre of this area being at the centre of the slab. Then the maximum moment will occur at the centre of the span, and may be considered distributed over a width b_m given by the expression,

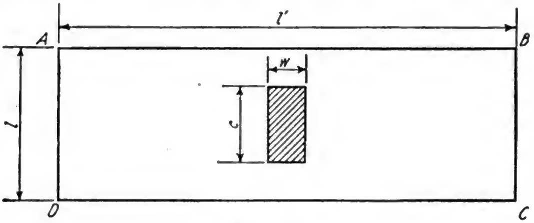


FIG. 37y.

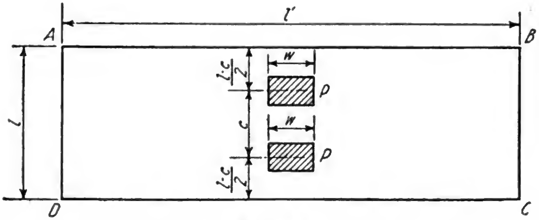


FIG. 37z.

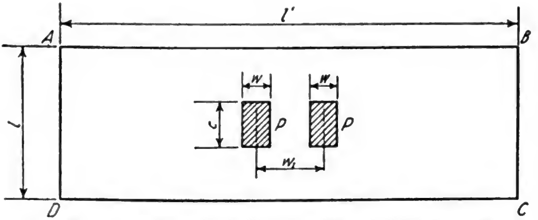


FIG. 37aa. Arrangements of Concentrated Loads on Slabs.

$$b_m = w + l \tan \theta, \text{ but never } > l'. \qquad \text{[Eq. 149]}$$

If we should enter Fig. 37x with values of $\frac{l'-w}{l}$, rather than those of $\frac{l'}{l'}$, the width b_m would always come out less than l', and this procedure would probably be the more logical one to follow; but the refinement would hardly be warranted. The moment per unit of width at the centre of the span is evidently

$$M = \frac{P}{4b_m}\left(l - \frac{c}{2}\right). \qquad \text{[Eq. 150]}$$

If a slab simply supported along two sides carries two equal loads P arranged symmetrical y about the centre of the span, as shown in Fig. 37z, we may consider the value of b_m to be

$$b_m = w + (l - c) \tan \theta, \text{ but never} > l' \qquad \text{[Eq. 151]}$$

In this case it would be logical to employ values of $\frac{l'-w}{l-c}$ instead of those for $\frac{l'}{l}$; but, as in the preceding case, the use of the ratio $\frac{l'}{l}$ is sufficiently accurate. The moment per unit of width is evidently

$$M = \frac{P}{2b_m}\left(l - c\right). \qquad \text{[Eq. 152]}$$

If a similar slab carries two equal loads P, arranged as shown in Fig. 37*aa* and located symmetrically about the centre of the slab, the width b_m should first be found for one of the loads by Equation 149. If the resulting value of b_m is not greater than w_1, it 's to be used; but if it is larger, the correct value of b_m for the two loads can be found by the formula,

$$b_m = w + w_1 + l \tan \theta, \text{ but never} > l'; \qquad \text{[Eq. 153]}$$

and the value of the moment per unit of width is then evidently

$$M = \frac{P}{2b_m}\left(l - \frac{c}{2}\right). \qquad \text{[Eq. 154]}$$

Ordinarily, a slab will be supported along the lines AD and BC as well as along the lines AB and CD. In case the ratio $\frac{l'}{l}$ equals or exceeds two, the formulæ already given should be used unchanged. When this ratio is less than two, the effect of the additional supports can be allowed for, either by increasing the value of tan θ somewhat or else by assuming a portion of the load to be carried to the said additional supports. The latter method is to be preferred, especially if the ratio $\frac{l'}{l}$ is considerably less than two. The division of loads between two systems of reinforcement in rectangular slabs supported on four sides has already been discussed.

If a slab is restrained at the supports, the effective width b_m will be less than for a slab which is simply supported. Approximate values of $\tan \theta$ for such slabs may be determined as follows:

In the case of a slab fixed at both ends, there will be lines of contraflexure at the quarter points of the span, so that the centre portion is practically a simple beam of length $0.5l$. If the slab should remain straight along these lines of contraflexure, the proper distribution width would be one-half of that for a simply supported span. However, points on these lines directly opposite the loads will deflect more than points on either side, which will increase somewhat the width of distribution. In the absence of any definite information, it will be satisfactory to assume a value of $\tan \theta$ about three-quarters of that for the simply supported slab. The lower curve on Fig. 37x has been drawn on this basis. For small values of the ratio $\frac{l'}{l}$ it has been assumed that the distribution would be about the same as for a simply supported slab.

For a slab fixed at one end and simply supported at the other, the value of $\tan \theta$ should be about the average of those for the slab fixed at both ends and for the slab free at both ends. The curve on Fig. 37x was drawn on this assumption.

A similar study for a span continuous over both supports with other spans of equal length indicates that the value of $\tan \theta$ will be about the same as for the slab fixed at one end and free at the other, and the curve for the latter condition on Fig. 37x will handle this case also. For a slab having one end free and the other continuous, the values should be taken as half-way between the curve last mentioned and that for a slab with both ends free.

The moment at the centre or ends of a slab fixed at both ends should be taken as one-half of that at the centre of a simply supported slab. In the case of a span fixed at one end and simply supported at the other, the above ratio should be ⅔; and this same value can be used with sufficient accuracy when one or both ends are continuous.

The effective width to be employed in figuring shearing stresses is not known positively; but for a load in any given position it seems quite certain that it should be at least as great as that effective for resisting the moment. In computing the maximum shear from the loading shown in Fig. 37y, the load should be placed so that the distance from the edge of the load area to the edge of the support is equal to the depth of the slab. If we let c_1 equal the distance from the centre of the support to the edge of the load area, we have for the value of the reaction on the said support

$$R = P \frac{l - \frac{c_1}{2} - d - \frac{c}{2}}{l}. \qquad \text{[Eq. 155]}$$

This value of R is also equal to the total shear on the section at the edge of the support. The effective width b_s for resisting this shear will then be that given by the equation,

$$b_s = w + 2\left(\frac{c_1}{2} + d + \frac{c}{2}\right) \tan \theta. \qquad \text{[Eq. 156]}$$

The value of $\tan \theta$ should be taken as ¾, unless the width of the slab is but little greater than w. The shear per unit of width is evidently equal to $\frac{R}{b_s}$.

In determining the shears produced by the loading shown in Fig. 37*z*, Equations 155 and 156 should be applied to each of the loads separately, and the sum of the results used. For the loading shown in Fig. 37*aa*, Equation 156 can nearly always be employed, as the value of b_s will be less than w_1 in practically every instance.

The tests reported by Messrs. Goldbeck and Slater seem to indicate that at working loads the width of distribution is independent of the amount of the transverse reinforcement. This effect is due to the tensile strength of the concrete, which, however, is not relied upon in correct design. In order to provide for the distribution given by the above formulæ, the transverse reinforcement should be from one-quarter to one-third as strong as the longitudinal.

The term "load area" has been used throughout this discussion to signify the area over which the load is applied to the upper surface of the slab. Each dimension of this "load area" is to be taken as equal to the corresponding dimension of the load itself, plus the thickness of any paving, filling, etc., which may intervene between the load and the slab. In the case of an electric railway track with the rails resting on steel ties embedded in a plain concrete slab, the "load area" for each axle-load should be taken about two (2) feet long, and equal in width to the tie length; while if the ties rest in ballast, a still greater area can be used.

THE CALCULATION OF STRESSES IN COLUMN FOOTINGS

The stresses in spread footings of columns are indeterminate. Quite a number of methods of calculation have been proposed, based on judgment or on approximate theoretical investigations. The only extensive experiments upon the subject that have ever been made were performed at the Engineering Experiment Station of the University of Illinois; and a discussion thereof by Prof. A. N. Talbot is to be found in Bulletin No. 67 issued by that school. It gives formulæ for design based upon the tests and theoretical studies.

The formulæ are derived for square footings of the type shown in Fig. 37*bb*. In this figure, l is the length of each side of the footing, which projects a distance *c* beyond the column in each direction; and *a* is the length of each side of the square column shaft. Prof. Talbot suggests that the moment in the footings be figured along a line $A - A'$ through one face of the column. He assumes the total moment on this section to be due to the load on one-quarter of the footing outside of the column, and finds for its value the expression,

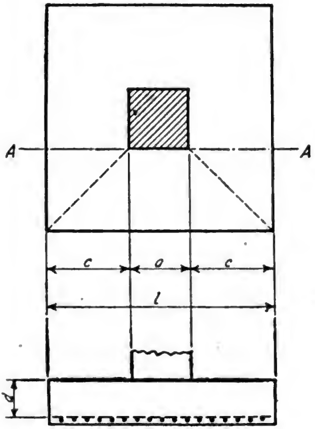


FIG. 37*bb*. Square Column Footing.

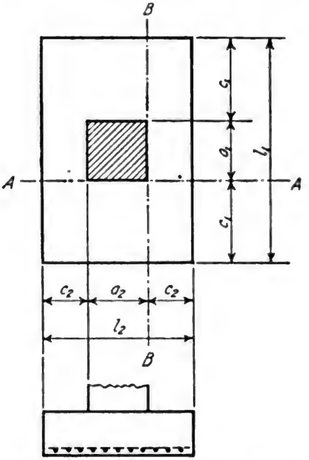


FIG. 37*cc*. Rectangular Column Footing.

$$M = (\tfrac{1}{2}\, a\, c^2 + 0.6\, c^3)\, w, \qquad \text{[Eq. 157]}$$

where *w* is equal to the foundation pressure per unit area less the weight of the footing slab per unit area. If the load on the trapezoid indicated by the dotted lines had been used, the coefficient 0.6 in the above equation would have been replaced by ⅔. The value 0.6 was arrived at by a theoretical consideration of the distribution of the load on the corner portions of the footing, based on the flexure curves from actual tests. Prof. Talbot then assumes this moment to be uniformly distributed over a width *b* given by the expression,

$$b = a + 2\, d + \tfrac{1}{2}\, (l - a - 2\, d). \qquad \text{[Eq. 158]}$$

The author does not consider that Equation 157 gives the correct value of the total moment on the section $A - A'$, and uses instead the formula,

$$M = \tfrac{1}{2}\, l\, c^2\, w. \qquad \text{[Eq. 159]}$$

This latter expression would certainly be the correct one if a wall footing, in which the wall extended along the line $A - A'$ for the full width of the footing, were under consideration; and there is no reason why the value should be any less in the case of the column footing. It is true that half of the load on the corner portion of the footing will be carried in the first place by reinforcement parallel to the section $A - A'$; but it must be eventually transferred to the column shaft by the reinforcement running in the other direction. The use of Equation 159 does not mean that the load at the corners is counted twice, as is stated on page 14 of the bulletin.

It is necessary to determine the effective width for resisting the moment given by Equation 159. An approximate value can be computed from Prof. Talbot's tests in the following manner:

For the unreinforced wall footings, the average ratio of the modulus of rupture of footing to that of control beam is 1.25. If for the unreinforced column footings the modulus of rupture be computed by assuming the moment given by Equation 159 to be uniformly distributed over the full width of the section, the corresponding value is found to be 0.87. Since $\frac{0.87}{1.25}$ equals 0.70, it is evident that about 70 per cent of the width of the column footing was effective for resisting the moment. This value is not to be considered as very accurate, because the results of the different tests were somewhat variable; but, undoubtedly, the correct value lies somewhere between 65 and 80 per cent.

For the reinforced wall footings which gave tension failures, the ratios of the computed steel stresses at the critical loads to the yield points of the steel average about 1.11. Most of these footings carried loads considerably above the critical values before final failure; but it is doubtful if the steel stresses were much greater than those at the critical loads. If for the reinforced column footings which failed in tension the moments be figured by means of Equation 159, and the stresses in the steel then be calculated on the assumption that the entire amount thereof is effective, the value of the above-mentioned ratio is found to be about 1.13. This result would indicate that at the time of failure practically the entire amount of steel was effective. As pointed out on page 94 of the Bulletin, however, it is probable that the bars near the centre of the footing reached their yield point at loads somewhat below the failure loads, and that as they gave way the distribution of the steel stresses became more nearly uniform. Undoubtedly, the effective width was somewhat less than the total before the failure stage was reached; and at working loads it would evidently be still smaller. Taking into consideration the results obtained from the unreinforced footings, it would appear that the effective width was probably somewhat less than seven-eighths ($\frac{7}{8}$) of the total—say from eighty to eighty-five per cent.

The effective widths previously discussed have referred only to those for the steel. For the concrete stresses, the proper values will doubtless be less; but as the shear (diagonal tension) will always determine the thickness of the slab, the compressive stresses are of no importance.

For designing purposes it will be necessary to assume a formula for the effective width that will give consistent results for footings of various proportions. A study of the results of the tests fails to give much information concerning the proper form of such an equation. It does indicate, however, that the depth of the footing is of small importance, the effective widths for thin footings seeming to be as great as those for much thicker ones. Considering the question from the theoretical standpoint, it is evident that the ratio of the dimensions of the column shaft to those of the footing itself will be the most important factor. In the tests under consideration, this ratio was one-fifth ($1/5$); and the effective width was probably eighty or eighty-five per cent of the total. When the ratio of the length of the side of the column shaft to that of the side of the footing exceeds $1/2$, it is quite certain that practically the entire width of the footing will be effective; and if the said ratio approaches zero, the effective width will reduce to, say, $2/3$ of the total. In view of the above, the author suggests that the value of the effective width b be taken as that given by the formula,

$$b = l\left(0.7 + 0.5\frac{a}{l}\right), \text{ but not } > l, \qquad \text{[Eq. 160]}$$

in which a and l have the meanings previously assigned to them.

Equations 159 and 160 enable the moment per foot width to be computed; and from this moment the depth and the steel area per foot width can be determined. This steel area per foot width must be maintained over the entire footing, not merely for the width given by Equation 160; for that equation was deduced on this assumption. Theoretically, it would appear to be better to space the steel more closely under the column shaft, and more widely near the edges; but Prof. Talbot's tests indicate that the uniform spacing gives just as good results, and, as it is the simpler, it should be adopted.

The stresses in rectangular footings may not follow exactly the same laws as those in square ones; but until further experimental data are available, it will be best to use the formulæ given for square footings.

For unreinforced footings, the moments and the effective widths should be figured as for the reinforced footings.

Prof. Talbot suggests that the diagonal tension should be calculated on a section at a distance d from the face of the column shaft. The considerations which determine the choice of this section have already been treated in this chapter, under the heading "Shear and Diagonal Tension." As is there stated, the author prefers to take the critical section at a distance $\frac{d}{2}$ from the edge of the column shaft. For a square base, the total shear on this section is given by the formula,

$$V = w\,[l^2 - (a + d)^2], \qquad \text{[Eq. 161]}$$

and the unit shear by the expression,

$$v = \frac{V}{4\,(a + d)\,j\,d}. \qquad \text{[Eq. 162]}$$

In the case of a rectangular footing of the outline shown in Fig. 37*cc*, we have, for the section distant $\frac{d}{2}$ from $A - A$,

$$V = \left(\frac{l_2 + a_2 + d}{2}\right)\left(c_1 - \frac{d}{2}\right) w, \qquad \text{[Eq. 163]}$$

and

$$v = \frac{V}{(a_2 + d)\,j\,d}. \qquad \text{[Eq. 164]}$$

The bond stress should be computed on the section through the face of the column, using the same assumptions as were made in figuring the tensile stress in the steel. For a square footing, therefore, we have as the shear per unit width on the section,

$$V = \frac{c\,l\,w}{b}, \qquad \text{[Eq. 165]}$$

in which b has the value given by Equation 160.
The unit bond stress is then given by the formula,

$$u = \frac{c\,l\,w}{b\,j\,d\,\Sigma\,o}, \qquad \text{[Eq. 166]}$$

where $\Sigma\,o$ represents the sum of the perimeters of the bars in a unit width. For the rectangular footing shown in Fig. 37*cc*, the corresponding formulæ are self-evident.

THE CALCULATION OF STRESSES IN WALL FOOTINGS

The calculation of stresses in wall footings is quite simple, after the positions of the critical sections for bending moment and diagonal tensions have been determined. An extensive series of tests bearing on this question was reported in Bulletin No. 67 of the Engineering Experiment Station of the University of Illinois, to which reference has been made in the preceding section of this chapter; and from these tests the locations of the critical sections can be readily determined.

The bending moment should be figured on the section at the face of the wall. If we let c be the projection of the wall and w the difference between the soil pressure and the weight of the footing per unit area, the moment per unit length is given by the formula

$$M = \tfrac{1}{2}\,c^2\,w. \qquad \text{[Eq. 167]}$$

As explained in this chapter in the section entitled "Shear and Diagonal Tension," the critical section for diagonal tension is to be taken at a distance from the face of the wall equal to the depth of the slab. The shear per unit width on this section is given by the formula

$$V = w\,(c - d). \qquad \text{[Eq. 168]}$$

The critical section for pure shear is evidently at the face of the wall. The value of the shear per unit width at this section can be found by the formula

$$V = w\,c \qquad \text{[Eq. 169]}$$

The bond stress should also be figured on the section at the face of the wall, using the shear given by Equation 169.

Equations 167, 168, and 169 assume that the soil pressure is constant over the entire base. If it varies, this fact must be properly considered.

The equations given above evidently apply to plain footings as well as to reinforced ones.

THE CALCULATION OF STRESSES IN ARCH RIBS WITH FIXED ENDS

It has been the author's practice to use the elastic theory for the design of fixed-ended reinforced concrete arch ribs. Convenient formulæ, developed in accordance with the assumptions of this theory, are to be found in Part II of "Modern Framed Structures," page 169 *et seq.*, in Turneaure and Maurer's "Principles of Reinforced Concrete Construction," page 335 *et seq.*, and in Taylor and Thompson's "Concrete, Plain and Reinforced," page 551 *et seq.*, the chapter on arches in the latter book being the work of Prof. F. P. McKibben. The formulæ given by Turneaure and Maurer and by Taylor and Thompson assume that the rib is divided into such lengths $d\,s$ that the ratio $\frac{d\,s}{I}$ will remain constant. While this simplifies the formulæ, it causes the lengths $d\,s$ adjacent to the springing to be undesirably long, unless the number of divisions of the ring be made excessively great. In designs made in the author's office, it has been customary to divide the horizontal projection of the arch rib into parts of equal length—there being usually twenty such divisions in the whole arch, or ten in each half. The formulæ given in "Modern Framed Structures" have been used, but the summations have been taken as explained by Turneaure and Maurer and by Taylor and Thompson, instead of for the full length of the rib in all cases. This change requires the placing of the coefficient 2 before certain of the terms of the equations given in "Modern Framed Structures," as will be noted by comparing them with Equations 194, 195, 196, and 212 of this chapter.

Just before the manuscript of this chapter was sent to the printer, Part III of Hool's "Reinforced Concrete Construction" was received by the author. It gives a very extensive treatment of arch analysis by the elastic theory, presenting both the ordinary method of solution and a graphical method devised by A. C. Janni, Esq., C.E., and termed the Method of the Ellipse of Elasticity. This new method is claimed to save a considerable amount of time; but the author has not yet had an opportunity to check the correctness of the said claim. The effect of elastic piers is also treated, both by the usual method of solution and by the Method of the Ellipse of Elasticity.

The method of laying out the centre line of an arch rib will first be discussed, and then the equations to be used in its preliminary design and in its final design; after which the application of these principles to the calculation of the stresses in an arch will be illustrated.

Determination of the Centre Line of Rib

The centre line of an arch rib should, preferably, be so located that the maximum moments at the various sections will be as small as possible, as this will permit the use of the minimum section of rib, and will require the least amount of reinforcement. The form to be adopted for any particular case will depend upon the distribution of the loading, and also upon the ratio of the rise to the span length. Occasionally the outline of a rib may be determined by special conditions, such as underclearance requirements, or a particular curve may be used for æsthetic purposes; but a rib laid out in this manner is likely to be uneconomic.

In the following discussion moments will be considered positive when causing compression in the upper fibres of a rib, and negative when causing compression in the lower fibres thereof.

For an arch in which the rise is high as compared with the span length, the moments at various sections will be mainly those produced by partial live loads. Temperature stresses will not be large; and they will have no effect on the section of the rib required, since an increase in unit stresses of thirty (30) per cent is allowed when they are considered. The moments from arch shortening will also be small. For such an arch it will be best to make the centre line of the rib follow the funicular polygon for dead load plus one-half live load over the entire span. When this is done, the moments at all sections due to the above loading will be practically zero, if we neglect the effect of arch shortening. In order to produce a maximum positive moment at any given section we shall vary from this condition of loading by adding half live-load concentrations at certain load points and removing equal amounts at all the other load points; while to cause a maximum negative moment at the same section we shall remove half live-load concentrations at the first-mentioned load points, and add equal loads at the other load points. Evidently, therefore, the maximum positive and negative moments at any section will be practically equal, which is the most favorable condition possible. Owing to the effect of arch shortening, the positive moments will exceed the negative a trifle at the crown, and the negative will be a little greater than the positive at the springing; but it will be found impossible to alter the shape of the centre line of the rib so as to reduce the maximum moments at one of these points, without at the same time increasing that at the other. Furthermore, as a point in the haunches, at which the stresses due to arch shortening are even smaller than at the crown, will usually be the critical section, the obtaining of an exact balance of moments at the crown and springing is of no importance.

In a flat arch the stresses from arch shortening and temperature are very important. Arch shortening produces positive moments at the crown and negative moments at the springing, and a fall of temperature produces the same effects. The fall of temperature is generally assumed to be about 50 degrees F., and the rise, 30 degrees F. Since the effect of arch shortening is in most spans about the same as that of a 15 degrees F. fall of temperature, the combination of the two effects will produce the same stresses as a 65 degrees F. fall of temperature, or a 15 degrees F. rise. This will result in high positive moments at the crown, and high negative moments at the springing. These effects can be balanced to some extent by laying out the centre line of the rib to follow the funicular polygon for dead load only. Under these conditions, the dead-load moments at all sections will be practically zero; and since the positive live-load moments will be a trifle greater than the negative at the crown, and considerably greater at the springing, the positive dead-plus-live-load moments at these sections will be greater than the negative, rather than equal to them as in the case of the rib laid out for dead-plus-half-live loads. This increase in positive moment at the crown is not desirable, but its amount is small, and the effect thereof is of little importance. The decrease in the negative moment at the springing is an advantage, as it will usually permit a reduction in the section at that point.

There is a further point in favor of laying out the centre line of the rib for the dead load only, rather than for the dead load plus half of the live load, in that it ensures that the line of pressure will be closer to the centre line of the rib under the condition of loading generally prevailing.

The ratio of rise to span length at which it is immaterial whether the centre line be laid out for dead load only, or for dead load plus half live load, depends to some extent upon the loading and the form of the arch rib. For comparatively thick ribs the critical value will be larger than for thinner ones; while for great live-load moments the said critical value will be less than for smaller live-load moments. Ordinarily it may be expected to be from one-fifth ($^1/_5$) to one-sixth ($^1/_6$). For a considerable range of values, it will make but little difference which of the methods is employed. For any particular case the question can be settled by means of the equations given later for the approximate design of arch ribs.

The usual method of laying out the centre line of an arch rib is to determine graphically the funicular polygon for the assumed condition of loading, and then locate the centre line to follow it as closely as possible. A method which has been used in the author's office for several years is believed to be preferable, and is accordingly given below. It was derived by E. A. Slettum, Esq., C.E., and C. W. Yelm, Esq., C.E., while they were in the employ of the author's firm. A brief description of this method is to be found in the before-mentioned paper by E. E. Howard, Mem. A. S. C. E., entitled "The Twelfth Street Trafficway Viaduct, Kansas City, Mo." and printed in the *Proc.* A. S. C. E. for May, 1915. Similar methods have been used by several other writers.

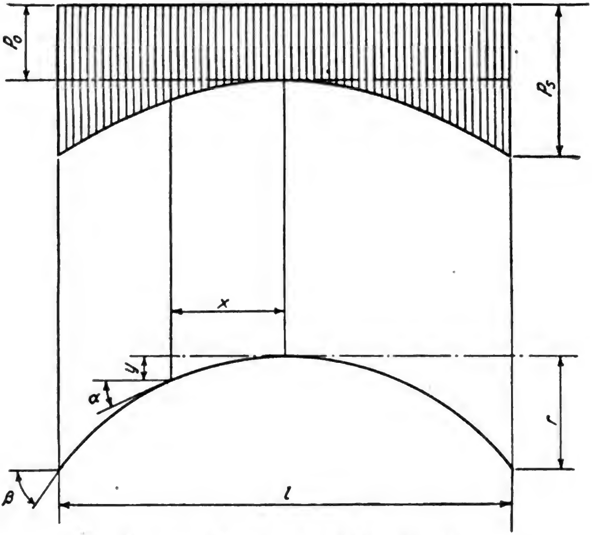


FIG. 37*dd*. Sketch of Arch Rib and Loading.

Suppose the centre line of the arch rib shown in Fig. 37*dd* to coincide with the funicular polygon for the varying load represented by the shaded area at the top of the figure, the curved line at the bottom of this area being a parabola with its vertex at the centre of the span. Let p_o be the load per lineal unit at the crown, and p_s that at the springing. The thrust H_o at the crown of the arch can be found by taking moments about the springing, and is evidently

$$H_o = \frac{1}{r}\left[\frac{1}{8}\,p_o\,l^2 + \frac{1}{48}\,(p_s - p_o)\,l^2\right] = \frac{l^2}{48\,r}\,(p_s + 5\,p_o). \qquad \text{[Eq. 170]}$$

Now the slope of the arch rib, $\frac{dy}{dx}$ or $\tan \alpha$, at any point distant x from the crown must be equal to the load on the rib between the said point and the crown divided by H_o, so that we may write

$$\frac{dy}{dx} = \tan \alpha = \frac{p_o\, x + \frac{1}{3}(p_s - p_o)\frac{4\, x^2}{l^2}\, x}{\frac{l^2}{48\, r}(p_s + 5\, p_o)}. \qquad \text{[Eq. 171]}$$

On dividing both numerator and denominator of the right-hand member of this equation through by p_o, and letting $\frac{p_s}{p_o} = u$, we have

$$\frac{dy}{dx} = \tan \alpha = \frac{x + \frac{4}{3}(u-1)\frac{x^3}{l^2}}{\frac{l^2}{48\, r}(u + 5)} = \frac{4\, r}{l^2\,(u+5)}\left[12\, x + 16\,(u-1)\frac{x^3}{l^2}\right]. \qquad \text{[Eq. 172]}$$

Integrating this latter expression, we get

$$y = \frac{4\, r}{l^2\,(u + 5)}\left[6\, x^2 + (u - 1)\frac{4\, x^4}{l^2}\right]. \qquad \text{[Eq. 173]}$$

This last formula is the desired equation for the centre line of the rib.

The assumption made above, that the load on the rib will vary with the ordinates of a parabola, has been checked against several arches, and has been found to be very reliable. It applies for both open-spandrel and solid-spandrel arches.

It is necessary to divide the arch rib into a number of sections in order to apply approximate methods of integration, and for this purpose the above equations for y and $\frac{dy}{dx}$ can be put into more convenient forms. Suppose the horizontal projection of one-half of the rib to be divided into N_s equal parts, and let the crown be numbered 0, and the other points of division, 1, 2, 3, etc., the number of the springing point being N_s. Let N represent the number of any point. Then since x for any point equals $\frac{N}{N_s} \times \frac{l}{2}$, the equations for y and $\frac{dy}{dx}$ become

$$y = \frac{r}{u + 5}\left[\frac{6\, N^2}{N_s^2} + (u - 1)\frac{N^4}{N_s^4}\right], \qquad \text{[Eq. 174]}$$

and $$\frac{dy}{dx} = \tan \alpha = \frac{r}{l\,(u + 5)}\left[\frac{24\, N}{N_s} + (u - 1)\frac{8\, N^3}{N_s^3}\right]. \qquad \text{[Eq. 175]}$$

It will usually be convenient to let N_s equal twenty (20), in which case these equations take the forms

$$y = \frac{r}{u+5}\left[\frac{3N^2}{200} + \frac{(u-1)}{16}\,\frac{N^4}{(10)^4}\right], \qquad \text{[Eq. 176]}$$

$$\text{and } \frac{dy}{dx} = \tan\alpha = \frac{r}{l\,(u+5)}\left[\frac{12N}{10} + (u-1)\,\frac{N^3}{(10)^3}\right]. \qquad \text{[Eq. 177]}$$

It will be noted that the method outlined takes care of vertical loads only. In a spandrel-filled arch of considerable rise, the horizontal pressure of the filling must also be considered. This will require the rib to be curved more sharply near the ends, which result can be obtained by increasing the value of u. If the horizontal forces are of importance, it will be best to determine the position of the centre line of the rib graphically.

For flat arches the curve represented by Equation 173 is often practically a segment of a circle. For such arches it will be well to test this relationship; and if the agreement is close the circular curve should be used, as the work of laying out the rib will be simplified thereby.

The ellipse is frequently employed as the curve of the intrados of an arch rib, especially in the case of spandrel-filled arches. This curve is not suitable for the centre line of a rib, as it is too highly inclined at the springing. However, by thickening the rib considerably just at the springing the intrados can be made an ellipse without the centre line itself deviating unduly from the proper curve. It will still probably have too sharp a curvature at the ends, so that the positive moments at the springing from dead and live loads will be much larger than the negative. The positive moments at the crown will be a trifle greater than the negative, and the negative a little greater than the positive in the haunches. The excess of positive moment at the springing will be partly offset by the negative moments from arch shortening and fall of temperature. These latter effects will, however, produce positive moments at the crown, and the thickness at that point may have to be increased a trifle for this reason. The large increase of thickness required at the springing will not affect the appearance of a spandrel-filled arch, as it need not show on the outer surface.

Occasionally it is necessary to place one springing of an arch at a higher elevation than the other. This results in an unsymmetrical rib. If the difference is small, the rib can be designed as though it were symmetrical, with the springings located at an elevation which is the average of the ones used for the two actual springings. In laying out the rib, the x's of the various points are then to be measured horizontally and the y's are to be measured vertically downward from a line drawn through the crown parallel to the line joining the actual springings.

Equation for Thickness of Rib

It is desirable to have an equation for the thickness of the rib at any point in terms of the thickness at the crown and springing. This expression must be of such form that the thickness will increase gradually from the crown to the springing. A formula which can be used is

$$h = h_o + \frac{2x}{l} \frac{(h_s - h_o)}{\sec \beta} \sec \alpha. \qquad \text{[Eq. 178]}$$

Introducing the quantity $\frac{l\,N}{2N_s}$ instead of x as before, we have

$$h = h_o + \frac{N}{N_s} \frac{(h_s - h_o)}{\sec \beta} \sec \alpha. \qquad \text{[Eq. 179]}$$

When $N_s = 20$, this equation becomes

$$h = h_o + \frac{N}{20} \frac{(h_s - h_o)}{\sec \beta} \sec \alpha. \qquad \text{[Eq. 180]}$$

Equations 178, 179, and 180 are not adapted to flat arch ribs, as they give a greater thickness than is required in the haunches, thus wasting concrete and increasing unnecessarily the temperature stresses and arch-shortening stresses at the crown and springing. For such ribs it will be best to replace the ratio $\frac{\sec \alpha}{\sec \beta}$ in these equations by its square or cube, or even a considerably higher power for very flat arches. Such a value should be chosen that the increase of thickness at the quarter-point of the span over that at the crown will be somewhat less than $\frac{1}{4}\ (h_s - h_o)$.

For a rib with an elliptical intrados, it is possible that none of the equations suggested above will serve, owing to the rapid increase in thickness required near the springing. For such a rib it will be best to use the form suggested in the preceding paragraph from the crown to a point near the springing, and then determine the thicknesses for the remainder by laying the rib out to scale.

Approximate Methods of Calculation

It is important to be able to compute quickly approximate values of the stresses at the critical sections of an arch rib, both for the purpose of making estimates and in order to select the sections of a rib to be used in a final design. Figs. 37*ee*, 37*ff*, and 37*gg*, and Equations 181 to 193, inclusive, will give results which are sufficiently accurate for these purposes. They were worked out from the actual designs of a number of arches. They apply directly to ribs laid out in the manner before explained, in which the thickness varies in accordance with Equation 178; and with a few modifications they will also serve for certain other forms, as will be explained later. They probably give results sufficiently accurate for final designs, since the stresses involved are indeterminate; but the author cannot guarantee this until they have been checked against a greater number of ribs.

The thrust at the crown of the arch, when it is loaded with the loading for which its centre line was laid out, is given by Equation 170. Putting $p_s = up_o$ in this equation, it becomes

$$H_o = \frac{p_o l^2}{48r}(u + 5). \qquad \text{[Eq. 181]}$$

The thrust at any other point of the rib, due to this same loading, is

$$T = H_o \sec \alpha; \qquad \text{[Eq. 182]}$$

and at the springing it has the value,

$$T_s = H_o \sec \beta. \qquad \text{[Eq. 183]}$$

The moment at any point of the rib, due to the loading for which it was laid out, is theoretically zero (neglecting for the present the effect of arch shortening). However, the said loading will usually not vary exactly as the ordinates of a parabola, and, therefore, there will be small moments at various sections; and if a portion of the load is concentrated at certain points, as in an open-spandrel arch, positive moments will occur at the load points and negative moments at the points midway between. An examination of the results for several arches indicates that the positive moment at or near any load concentration can be taken as equal to

$$M = + \frac{p_o l^2}{1500}, \qquad \text{[Eq. 184]}$$

and that the moments at other points can be assumed to be

$$M = \pm \frac{p_o l^2}{3000}. \qquad \text{[Eq. 185]}$$

The positive and negative live load moments (exclusive of the effect of arch shortening) at the crown, springing, and various intermediate points can be determined by the formula

$$M = C_m p l^2, \qquad \text{[Eq. 186]}$$

in which p is the total live-load per lineal foot, and C_m is a coefficient the value of which can be taken from the curves of Fig. 37*ee*. It will be noted that the value of C_m depends upon the ratio of the moment of inertia at the springing to that at the crown.

When the centre line of the rib is laid out for dead load only, the maximum moment at any section due to dead and live loads only will be the sum of the live-load moment as determined by Equation 186 and that given by Equation 184 or Equation 185. The live-load thrust at the crown can be considered to be that due to half live load over the entire span, and its value can be taken as

$$H_o = \frac{p}{2} \times \frac{l^2}{8} \times \frac{1}{r} = \frac{p\,l^2}{16r} \qquad \text{[Eq. 187]}$$

The live-load thrusts at other points can be figured with sufficient accuracy by the use of Equations 182 and 183.

When an arch rib is laid out for dead load plus one-half live load, the moment at any section under this loading is given by Equation 184 or Equation 185 (neglecting, as before, the effect of arch shortening).

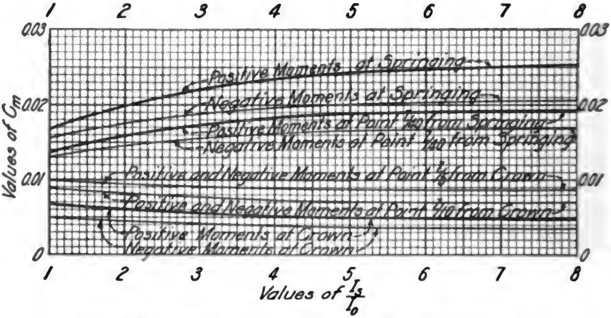


FIG. 37*ee*. Values of Live-Load Moment Coefficient C_m.

The maximum moment at the same point due to combined dead and live loads can be found by adding to this value the average of the positive and negative live-load moments at the section, as determined by Equation 186 and Fig. 37*ee*. This added moment may be either positive or negative. The thrust at the crown is given by Equation 181; and the thrusts at other sections can be figured by means of Equations 182 and 183.

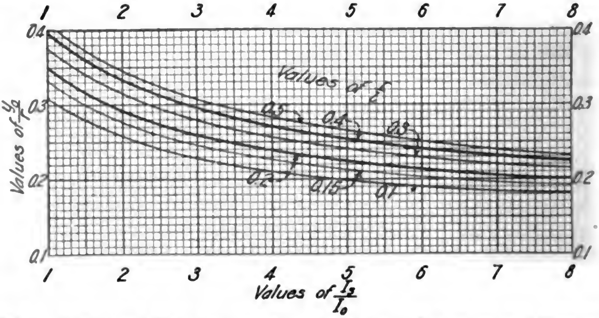


FIG. 37*ff*. Position of Point of Contraflexure for Arch Shortening and Temperature Stresses.

The position of the plane of contraflexure for arch shortening and temperature stresses can be found by means of Fig. 37*ff*, which gives the ratio of the distance y_0 that the said plane lies below the crown to the rise r. As will be noted, the value of $\frac{y_o}{r}$ depends upon the ratio of the moment of inertia at the springing to that at the crown, and also on the ratio of the rise r to the span length l.

The thrust at the crown due to arch shortening can be figured by the formula,

$$H_a = -\left(10 + 3\frac{I_s}{I_o}\right)\frac{H_o I_o}{r^2 A_o \sec\beta} = -\left(1 + 0.3\frac{I_s}{I_o}\right)\frac{H_o h_o^2}{r^2 \sec\beta}. \qquad \text{[Eq. 188]}$$

The last member of this equation is written on the assumption that I_o equals $\frac{A_o h_o^2}{10}$, which is about true for ordinary percentages of reinforcement, as can be seen by referring to Fig. 37t.

The moment at the crown from arch shortening is

$$M_a = -H_a y_o. \qquad \text{[Eq. 189]}$$

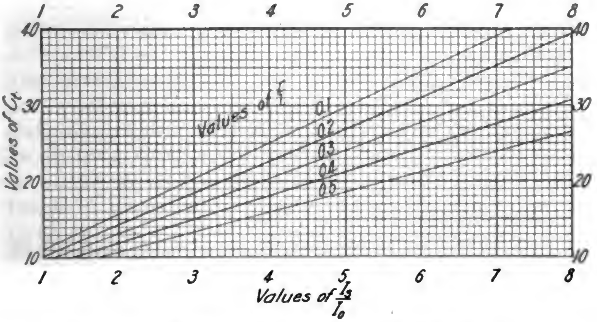


FIG. 37gg. Values of Temperature Stress Coefficient C_t.

M_a is always positive, since H_a is always negative. The moment at any other point is equal to

$$M = H_a(y - y_o); \qquad \text{[Eq. 190]}$$

and at the springing it has the value,

$$M_s = H_a(r - y_o). \qquad \text{[Eq. 191]}$$

The moment at the springing is always negative. The thrust T at any section due to arch shortening is given by the formula

$$T = H_a \cos\alpha. \qquad \text{[Eq. 192]}$$

It is always negative.

The thrust at the crown due to a change of temperature is given by the formula,

$$H_t = C_t \frac{\omega t E I_o}{r^2}, \qquad \text{[Eq. 193]}$$

the values of the coefficient C_t being taken from the curves of Fig. 37*gg*. Its value depends on those of the ratios $\frac{I_s}{I_o}$ and $\frac{r}{l}$. The quantity t carries its own sign, it being positive when the temperature rises and negative when it falls. The moments at various points are to be figured by means of Equations 189, 190, and 191, replacing M_a and H_a by M_t and H_t. For a rise of temperature the moment at the crown is negative, and that at the springing positive; while for a fall of temperature the signs are reversed. The thrust at any section is to be calculated by means of Equation 192, replacing H_a by H_t. It is positive when the temperature rises, and negative when it falls.

For a flat arch in which the centre line of the rib is laid out as already explained, but in which the thickness does not vary in accordance with Equation 178, the above formulæ can be applied as follows: Figure the moment of inertia at the quarter point, and then compute its value at the same point as though the thickness varied in accordance with Equation 178. Call the percentage that the first of these values falls below the second one x. Then the change in the thickness of the rib will cause a reduction in the arch shortening thrust of $0.4x$ per cent, a reduction in the thrust from temperature changes of $0.6x$ per cent, and an increase in the live-load moments at the crown and at points within $\frac{l}{10}$ thereof of $0.2x$ per cent. The live-load moments at other sections will be but slightly affected, as will also the position of the plane of contraflexure for arch shortening and temperature thrusts.

For an elliptical rib the positive moment at the crown will be a trifle greater than the value given by Fig. 37*ee*; the negative moment in the haunches will also be a trifle greater; and the positive moment at the springing will be considerably greater. The amounts of these changes can be found by laying off the centre line of the rib to scale, and then drawing in a funicular polygon for dead load plus half live load over the entire span which will pass just above the centre line at the crown, and just below it in the haunches. The moments due to the deviation of this funicular polygon from the centre line of the rib should then be computed, and added to the values found as previously explained for an arch laid out for dead load plus half of live and impact loads.

The formulæ and diagrams just given can be applied to an actual rib in the following manner, after the loads which come upon the rib and its rise and span have been determined:

1. Assume values of h_o, h_s, and sec β, and calculate p_o, p_s, and u, preferably for dead load plus one-half live load. The equivalent uniform live load should be taken for a loaded length of $\frac{l}{2}$.
2. Compute I_o, I_s, $\frac{I_s}{I_o}$, and $\frac{r}{l}$, and substitute the values of r, l, and u into Equations 172 and 173.
3. Figure the dead- and live-load stresses at the crown and springing (and at one or more points in the haunches for arches of high rise) by means of Equations 181 to 187, inclusive, and Fig. 37*ee*.
4. Compute the stresses from arch shortening at the same sections by means of Equations 188 to 192, inclusive, and Fig. 37*ff*
5. Compute the stresses from temperature changes at the same sections by means of Equations 193, 189, 190, 191, and 192, and Figs. 37*ff* and 37*gg*.
6. Test the various sections for the calculated stresses, and revise the assumed values as required.
7. Recalculate the stresses at the various sections if the loads and the ratio $\frac{I_s}{I_o}$ assumed at first were too much in error, and refigure the sections.

It is suggested above that the value of u be computed for dead load plus one-half live load, because that is the condition which will be used, unless some other condition is found to permit the adoption of smaller sections for the rib. The error involved will never be large, and will always be on the side of safety.

The value of p_s should not be computed for the actual loads at the springing section, since there will probably be very heavy details at that point for æsthetic reasons; but it should be figured on the assumption that the construction used over the arch itself is employed at this point also.

The value of sec β for a parabolic rib is $\sqrt{1 + \left(\frac{4r}{l}\right)^2}$. For ordinary ribs, it will be a few per cent greater than this, the exact expression being $\sqrt{1 + \left(8\frac{r}{l} \times \frac{u+2}{u+5}\right)^2}$. The value of sec α at the quarter point of a parabolic rib is $\sqrt{1 + \left(\frac{2r}{l}\right)^2}$; and for ordinary ribs it is about one per cent less.

To illustrate the procedure just outlined, suppose that we wish to make an approximate design for the arch ribs of a two-ribbed, open-spandrel arch of 93.3 feet span and 18.5 feet rise, which carries a roadway 28′ wide, the live load being Class *A*. The loads from the floor, etc., are supposed to have been figured previously. We proceed as follows:

$$\frac{r}{l} = \frac{18.5}{93.3} = 0.2.$$

$$\sec\beta = 1.3 \text{ say } (=1.28 \text{ for a parabolic rib}).$$

Assume each rib to be 4′ 4″ wide throughout, 2′ 0″ thick at the crown and 3′ 8″ thick at the springing. The thickness will be assumed to vary in accordance with Equation 178, its value at any point being given by the expression,

$$h = 24 + 20\frac{x}{46.7} \times \frac{\sec\alpha}{1.3} = 24'' + 0.33\,x\sec\alpha.$$

The values of p_o and p_s for dead load plus half of live and impact loads are then figured as follows:

p_o — for one rib.

D. L. Floor, paving, spandrel cols., etc.		3380 lbs. per lin. ft.
Arch rib, 2′ 0″ × 4′ 4″ × 150 lbs.		1300 " "
L. L. Cl.A., 47′ span	113 lbs. per sq. ft.	
Imp — $n = 2$ 34%	38 lbs. per sq. ft.	
½ (L. L. + I. L.)	151 × 14 × ½	1060 " "
Total		5740 lbs. per lin. ft.

p_s — for one rib.

D. L. Floor, paving, spandrel cols., etc	4410 lbs. per lin. ft.
Arch rib 3′ 8″ × 4′ 4″ × 150 × 1.3	3100 " "
½ (L. L. + I. L.) — as for p_o	1060 " "
Total	8570 lbs. per lin. ft.

$$\therefore u = \frac{8570}{5740} = 1.49.$$

Assume reinforcement as 0.5 per cent in each face throughout, located at a distance of $\frac{h}{10}$ from the surface.

$$\therefore I_o = 0.106 \times 4.33 \times 2 \times 2 \times 2 = 3.7 \quad \text{(Fig. 37}t\text{)},$$

$$I_s = 0.106 \times 4.33 \times 3.67 \times 3.67 \times 3.67 = 22.7 \quad \text{(Fig. 37}t\text{)},$$

and

$$\frac{I_s}{I_o} = 227 \div 3.7 = 6.1$$

Stresses from dead load plus one-half live load.

Crown.

$$H_o = \frac{5740 \times 93.3^2}{48 \times 18.5} \times 6.49 = +365{,}000 \text{ lbs.}$$

$$M_o = \frac{5740 \times 93.3^2}{1500} = +\ 33{,}000 \text{ ft. lbs.}$$

Haunch—say at a spandrel column located at a distance of $\frac{l}{10}$ from the crown.

$T =$ say $+\ 370{,}000$ ft. lbs.
$M =$ $+\ 33{,}000$ ft. lbs.

Springing.

$T_s = 365{,}000 \times 1.3 =$ $+\ 475{,}000$ lbs.

$$M_s = \pm \frac{5740 \times 93.3^2}{3000} = \qquad \pm\ 17{,}000 \text{ ft. lbs.}$$

Live-Load Moments.

Crown.

$M_o = \pm 0.0043 \times 2120 \times 93.3^2 =$ $\pm\ 79{,}000$ ft. lbs.

Haunch.

$M = \pm 0.007 \times 2120 \times 93.3^2 =$ $\pm\ 129{,}000$ ft. lbs.

Springing.

$M_s = \pm 0.0226 \times 2120 \times 93.3^2 =$ $\pm\ 417{,}000$ ft. lbs.

Stresses from arch shortening.

Crown.

$$H_a = (1 + 0.3 \times 6.1) \frac{2^2 H_o}{18.5^2 \times 1.3} = -\ 0.026 H_o = -9500 \text{ lbs.}$$

$y_o = 0.21 \times 18.5' = 3.9'$
$M_a = 9500 \times 3.9' =$ $= +\ 37{,}000$ ft. lbs.

Haunch.

$T =$ say $-\ 9000$ lbs.
$M = 9500(3.9 - 0.7)$ $= +\ 30{,}000$ ft. lbs.

Springing.

$T_s = -\ 9500 \div 1.3$ $= -\ 7000$ lbs.
$M_s = 9500(3.9 - 18.5)$ $= -\ 139{,}000$ ft. lbs.

Stresses from temperature changes.

Crown.

	50° Fall	30° Rise
$H_t = -\ 3\ 1 \times \dfrac{0.000006 \times 50 \times 288{,}000{,}000 \times 3.7}{18.5^2}$		
$=$	$-\ 29{,}000$ lbs.	$+\ 17{,}400$ lbs.
$M_t = 29{,}000 \times 3.9 =$	$+\ 114{,}000$ ft. lbs	$-\ 68{,}000$ ft. lbs.
Haunch.		
$T =$ say	$-\ 29{,}000$ lbs.	$+\ 17{,}000$ lbs.
$M = 29{,}000 \times 3.2 =$	$+\ 93{,}000$ ft. lbs.	$-\ 56{,}000$ ft. lbs.
Springing.		
$T_s = 29{,}000 \div 1.3 =$	$-\ 21{,}000$ lbs.	$+\ 13{,}000$ lbs.
$M_s = 29{,}000 \times 14.6 =$	$-\ 424{,}000$ ft. lbs.	$+\ 250{,}000$ ft. lbs

The three sections are now tested as follows:

Section at Crown.

	Temp. not Considered	Temp. Considered
Thrust.		
Dead + half live..........	+ 365,000 lbs.	+ 365,000 lbs.
Arch shortening...........	− 9,000 "	− 9,000 "
Temperature − 50° fall.....	....	− 29,000 "
Total..................	+ 356,000 lbs.	+ 327,000 lbs.
Moment.		
Dead + half live..........	+ 33,000 ft.-lbs.	+ 33,000 ft.-lbs.
Live load − positive.......	+ 79,000 "	+ 79,000 "
Arch shortening...........	+ 37,000 "	+ 37,000 "
Temperature − 50° fall.....	....	+ 114,000 "
Total..................	+ 149,000 ft.-lbs.	+ 263,000 ft.-lbs.
or	1,790,000 in.-lbs.	3,160,000 in.-lbs.
e = Mom. ÷ thrust	5″	9.7″
Dimensions $h \times b$	24″ × 52″ = 1248 sq. in.	1248 sq. in.
$\frac{h}{e}$	$\frac{24}{5} = 4.8$	$\frac{24}{9.7} = 2.5$
R/f_c....	$\frac{1,790,000}{52\times24^2\times600} = 0.10$	$\frac{3,160,000}{52\times24^2\times780} = 0.135$
$\frac{d'}{h}$ say............	$\frac{2.5}{24} = 0.1$	0.1
p per face (Fig. 37q)		0.57%

This is satisfactory.

Section in haunch—l/10 from crown.

	Temp. not Considered	Temp. Considered
Thrust.		
Dead + half live..........	+ 370,000 lbs.	+ 370,000 lbs.
Arch shortening...........	− 9,000 "	− 9,000 "
Temperature—50° fall......	....	− 29,000 "
Total..................	+ 361,000 lbs.	+ 332,000 lbs.
Moment.		
Dead + half live..........	+ 33,000 ft.-lbs.	+ 33,000 ft.-lbs.
Live load—positive........	+ 129,000 "	+ 129,000 "
Arch shortening...........	+ 30,000 "	+ 30,000 "
Temperature—50° fall......	....	+ 93,000 "
Total..................	+ 192,000 ft.-lbs.	+ 285,000 ft.-lbs.
=	2,300,000 in.-lbs.	3,420,000 in.-lbs.
e............................	6.4″	10.3″

	Temp. not Considered	Temp. Considered
Dimensions $h \times b$, $27'' \times 52''$	$= 1404$ sq. in.	1404 sq. in.
$\frac{h}{e}$	4.2	2.7
$\frac{d'}{h} = \frac{2.5}{27}$	$= 0.09$	0.09
p per face, $0.57 \times \frac{24}{27}$	$= 0.51\%$	0.51%
R/f_c (Fig. 37q)	0.120	0.131
f_c	510	690

Hence section as assumed is all right.

Section at springing.

Thrust.		
Dead + half live	+ 475,000 lbs.	+ 475,000 lbs.
Arch shortening	− 7,000 "	− 7,000 "
Temperature—50° fall	....	− 22,000 "
Total	+ 468,000 lbs.	+ 446,000 lbs.
Moment.		
Dead + half live	− 17,000 ft.-lbs.	− 17,000 ft.-lbs.
Live load—negative	− 417,000 "	− 417,000 "
Arch shortening	− 139,000 "	− 139,000 "
Temperature—50° fall	....	− 424,000 "
Total	− 573,000 ft.-lbs.	− 997,000 ft.-lbs.
	6,860,000 in.-lbs.	11,960,000 in.-lbs.
e	$14.7''$	$26.8''$
Dimensions $h \times b$, $44'' \times 52''$	$= 2290$ sq. in.	2290 sq. in.
$\frac{h}{e}$	3.0	1.64
R/f_c	$\frac{6{,}860{,}000}{52 \times 44^2 \times 600} = 0.113$	$\frac{11{,}960{,}000}{52 \times 44^2 \times 780} = 0.152$
$\frac{d'}{h} = \frac{2.5}{44}$	$= 0.06$	0.06
p per face (Fig. 37q)		0.6%

Hence section as assumed is all right.

Comparing the figures given above for the various sections, it is evident that the section at the point in the haunch is unnecessarily thick as compared with those at the crown and the springing. This was to have been expected, as Equation 178 is not suitable for so flat a rib. The thickness at the quarter point, taking the value of sec α a little less than $\sqrt{1 + (2 \times 0.2)^2}$, or 1.07, is

$$h = 24'' + 0.33 \times 23.3 \times 1.07 = 24'' + 8.2'' = 32.2''.$$

The increase over that of the crown has thus been 8.2″, while properly it should have been less than $\frac{20''}{4}$, or 5″. If the ratio $\frac{\sec\alpha}{\sec\beta}$ had been replaced by its square, the increase would evidently have been $8.2'' \times \frac{1.07}{1.3} = 8.2'' \times 0.82 = 6.7''$; if by the cube, $8.2'' \times 0.82^2 = 5.5''$; and if by the fourth power, $8.2 \times 0.82^3 = 4.5''$. Probably the last value would be the best. Adopting this, we get for the thickness at any point,

$$h = 24'' + 20\frac{x}{46.6}\left(\frac{\sec\alpha}{1.3}\right)^4 = 24'' + 0.15 \times \sec^4\alpha.$$

The thickness at the quarter point now becomes $24'' + 4.5'' = 28.5''$, rather than 32.2″. The ratio of the new moment of inertia to the former value is evidently $\left(\frac{28.5}{32.2}\right)^3$, or about 0.7. The moment of inertia at the quarter point has thus been reduced thirty (30) per cent. This will result in a reduction in the temperature stresses of $0.6 \times 30 = 18\%$, and in the arch shortening stresses of $0.4 \times 30 = 12\%$; and it will increase the live load moments near the crown by $0.2 \times 30 = 6\%$. The three sections will now be retested as follows, omitting the figures in which the action of temperature is not considered. as they are evidently unnecessary.

Section at crown.

Thrust.	
Dead + half live—as before	+ 365,000 lbs.
Arch shortening, − 9,000 × 0.88	− 8,000 "
Temperature—50° fall, − 29,000 × 0.82	− 24,000 "
Total	+ 333,000 lbs.
Moment.	
Dead + half live—as before	+ 33,000 ft.-lbs.
Live load—positive, + 79,000 × 1.06	+ 84,000 "
Arch shortening, + 37,000 × 0.88	+ 33,000 "
Temperature—50° fall, + 114,000 × 0.82	+ 94,000 "
Total	+ 244,000 ft.-lbs.
	or + 2,930,000 in.-lbs.
e	8.8″
Dimensions—$h \times b$	24″ × 52″ = 1248 sq. in.
h/e	2.7
$R/f_c = \frac{2,930,000}{52 \times 24^2 \times 780}$	0.125

$\frac{d'}{h} = \frac{2.5}{24}$	0.1
p per face (Fig. 37q)	0.45%
Section in haunch—l/10 from crown.	
Thrust.	
Dead + half live—as before	+ 370,000 lbs.
Arch shortening, say	− 8,000 "
Temperature—50° fall, say	− 24,000 "
Total	+ 338,000 lbs.
Moment.	
Dead + half live—as before	+ 33,000 ft.-lbs.
Live load—positive, + 129,000 × 1.06	+ 137,000 "
Arch shortening, 30,000 × 0.88	+ 26,000 "
Temperature—50° fall, 93,000 × 0.82	+ 76,000 "
Total	+ 272,000 ft.-lbs. or + 3,260,000 in.-lbs.
e	9.6″
Dimensions $h \times b$	25.4 × 52 = 1320 sq. in.
$h/e =$	2.7
$\frac{d'}{h} = \frac{2.5}{25.4}$	0.1
p per face, $0.45 \times \frac{24}{25.4}$	0.42%
R/f_c (Fig. 37q)	0.124
$f_c = \frac{3,260,000}{52 \times 25.4^2 \times 0.124}$	780
Section at Springing.	
Thrust.	
Dead + half live—as before	+ 475,000 lbs.
Arch shortening, −7,000 × 0.88	− 6,000 "
Temperature—50° fall, −22,000 × 0.82	− 18,000 "
Total	+ 451,000 lbs.
Moment.	
Dead + half live—as before	− 17,000 ft.-lbs.
Live load—negative—as before	− 417,000 "
Arch shortening, −139,000 × 0.88	− 122,000 "
Temperature—50° fall, −424,000 × 0.82	− 348,000 "
Total	− 904,000 ft.-lbs. or − 10,850,000 in.-lbs.
e	24.1″

Dimensions $h \times b$	$44 \times 52 = 2290$ sq. in
h/e	1.83
$R/f_c = \dfrac{10{,}850{,}000}{52 \times 44^2 \times 780}$	0.139
$\dfrac{d'}{h} = \dfrac{2.5}{44}$	0.06
p per face (Fig. 37*q*)	0.49%

From the preceding calculations it is evident that the thinner rib is the one which should be adopted. As the percentages of reinforcement required are low, a small reduction in the concrete sections at the crown and the springing would be permissible.

Exact Methods of Calculation

In the following exposition, it will be assumed that the arch rib is symmetrical about its centre line, as that is the condition which usually occurs. Should the rib be unsymmetrical, it will merely be necessary to form the summations of ds, $\dfrac{ds}{I}$, $\dfrac{y\,ds}{I}$, $\dfrac{y^2ds}{I}$, $\dfrac{x\,ds}{I}$, $\dfrac{x^2ds}{I}$, and $\dfrac{\sec\alpha\,ds}{A}$ for the entire rib, and drop the coefficient 2 wherever it occurs in Equations 194, 195, 196, and 212. The various summations involving M' will, of course, be different for the two halves of the rib; but the equations as written allow for this.

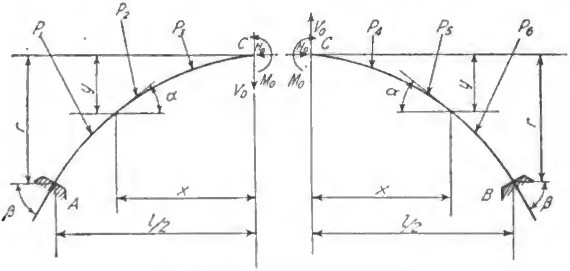


FIG. 37*hh*. Sketch of Arch Rib.

Let *A C B* in Fig. 37*hh* be an arch rib symmetrical about the crown *C*, fixed at the ends, and loaded in any manner. Suppose the rib to be cut at *C*, and let each half thereof be held in equilibrium by the stresses acting on the cut section. Then, using the notation given on page 789 *et seq.*, we have the following formulæ for the thrust, moment, and shear produced at the crown by external loads, neglecting the effect of arch shortening.

$$H_o = \frac{\int \frac{ds}{I} \int \frac{M'y\,ds}{I} - \int \frac{M'\,ds}{I} \int \frac{y\,ds}{I}}{2\left[\left(\int \frac{y\,ds}{I}\right)^2 - \int \frac{ds}{I} \int \frac{y^2\,ds}{I}\right]} \qquad \text{[Eq. 194]}$$

$$V_o = \frac{\int \frac{M'_l\,x\,ds}{I} - \int \frac{M'_r\,x\,ds}{I}}{2 \int \frac{x^2\,ds}{I}} \qquad \text{[Eq. 195]}$$

$$M_o = -\frac{2\,H_o \int \frac{y\,ds}{I} + \int \frac{M'\,ds}{I}}{2 \int \frac{ds}{I}} \qquad \text{[Eq. 196]}$$

The moment at any point in the left half of the rib is given by the equation,

$$M_l = M_l' + M_o + H_o\,y - V_o\,x; \qquad \text{[Eq. 197]}$$

and that at any point in the right half, by the formula,

$$M_r = M_r' + M_o + H_o\,y + V_o\,x. \qquad \text{[Eq. 198]}$$

The vertical components of the thrusts, the thrusts, and the shears due to vertical loads at any section are to be figured by the formulæ,

$$V_l = V_o + V'_l, \qquad \text{[Eq. 199]}$$
$$V_r = V_o - V'_r, \qquad \text{[Eq. 200]}$$
$$T_l = V_l \sin\alpha + H_o \cos\alpha, \qquad \text{[Eq. 201]}$$
$$T_r = -\,V_r \sin\alpha + H_o \cos\alpha, \qquad \text{[Eq. 202]}$$
$$S_l = V_l \cos\alpha - H_o \sin\alpha, \qquad \text{[Eq. 203]}$$

and

$$S_r = V_r \cos\alpha + H_o \sin\alpha. \qquad \text{[Eq. 204]}$$

In Equations 196 to 204, inclusive, H_o, V_o, and M_o carry their own signs, as determined from the preceding equations.

Should any of the loads be inclined, the quantity H_o is to be replaced in Equations 201 and 203 by $H_o + H_l$, and in Equations 202 and 204 by $H_o + H_r'$.

For arch shortening, we have the following formulæ:

$$H_a = -\frac{H_o \int \frac{\sec\alpha\,ds}{A} \int \frac{ds}{I}}{\sec\beta\left[\int \frac{ds}{I} \int \frac{y^2\,ds}{I} - \left(\int \frac{y\,ds}{I}\right)^2\right] + \int \frac{\sec\alpha\,ds}{A} \int \frac{ds}{I}}, \qquad \text{[Eq. 205]}$$

$$y_o = \frac{\int \frac{y\,ds}{I}}{\int \frac{ds}{I}}, \qquad \text{[Eq. 206]}$$

$$M_a = -H_a\, y_o, \qquad \text{[Eq. 207]}$$
$$M_l = M_r = M_a + H_a\, y = H_a\,(y - y_o), \qquad \text{[Eq. 208]}$$
$$T_l = T_r = H_a \cos \alpha, \qquad \text{[Eq. 209]}$$
$$S_l = -H_a \sin \alpha, \qquad \text{[Eq. 210]}$$
and
$$S_r = H_a \sin \alpha. \qquad \text{[Eq. 211]}$$

In Equation 205, H_o carries its own sign, as does H_a in Equations 207 to 211, inclusive. The second term in the denominator of Equation 205 amounts to about seven or eight per cent of the first term in arches having a rise of one-tenth of the span, to about two and one-half per cent when the rise is one-fifth of the span, and to less than one per cent when the rise is one-third of the span length. Hence it may be neglected except for flat arches, as the error thus produced is on the side of safety. For any particular case, the amount of this error will be about equal to $\frac{H_a}{H_o}$, as determined from Equation 188. When A varies about as sec α, the term $\int \frac{\sec \alpha\, ds}{A}$ can be replaced by $\frac{L}{2A_o}$; and when A varies nearly as $\sec^2 \alpha$, by $\frac{l}{2A_o}$.

For temperature stresses the following formulæ are to be used:

$$H_t = \frac{E\,\omega\, t\, l \sec \beta \int \frac{ds}{I}}{2 \sec \beta \left[\int \frac{ds}{I} \int \frac{y^2\, ds}{I} - \left(\int \frac{y ds}{I}\right)^2\right] + 2 \int \frac{\sec \alpha\, ds}{A} \int \frac{ds}{I}}, \qquad \text{[Eq. 212]}$$

$$M_t = -H_t\, y_o, \qquad \text{[Eq. 213]}$$
$$M_l = M_r = M_t + H_t\, y = H_t\,(y - y_o), \qquad \text{[Eq. 214]}$$
$$T_l = T_r = H_t \cos \alpha, \qquad \text{[Eq. 215]}$$
$$S_l = -H_t \sin \alpha, \qquad \text{[Eq. 216]}$$
and
$$S_r = H_t \sin \alpha. \qquad \text{[Eq. 217]}$$

The value of y_o is given by Equation 206. In Equation 212, t carries its own sign, as does H_t in Equations 213 to 217, inclusive. The equation for H_t can be simplified, for arches of considerable rise, by dropping the last term of the denominator of the right-hand member, as was explained above in the case of the equation for H_a.

The calculations should be carried out in the following manner, after the loads which come upon the rib, and its rise and span, have been determined:

1. Make a preliminary design in the manner previously outlined, and from it select the sections at the crown and springing, determine the law of variation of the rib thickness at intermediate points, and decide whether the centre line of the rib is to follow the polygon for dead only, or that for dead load plus half live load.

2. Recalculate p_o, p_s, and u, if necessary. Then decide upon the number of divisions into which the rib is to be divided, and determine the equations for y, $\frac{dy}{dx}$, and h.
3. Calculate the loads at the various load points, and check the value of u just selected by plotting the said loads to scale, as shown in Fig. 37*ii*. (The values of the various concentrations are, of course, proportional to the corresponding equivalent uniform loads.) If necessary, change the value of u, and determine the new equations for y, $\frac{dy}{dx}$, and h. With proper care it will never be necessary to recalculate the loads on account of such changes.
4. Calculate the quantities x, y, $\frac{dy}{dx}$, $\sec \alpha$, h, $h - 2\,d'$, $A_{conc.}$, A_{steel}, A, $I_{conc.}$, I_{steel}, I, ds, $\frac{ds}{I}$, $\frac{yds}{I}$, $\frac{y^2ds}{I}$, $\frac{xds}{I}$, $\frac{x^2ds}{I}$, and $\frac{\sec \alpha\, ds}{A}$ for the various sections, as indicated in Tables 37*b* and 37*c*, and form the summations for the quantities ds to $\frac{ds \sec \alpha}{A}$, inclusive.
5. Figure the values of H_o, V_o, and M_o for external loads in terms of the quantities $\int \frac{M'yds}{I}$, $\int \frac{M'xds}{I}$, and $\int \frac{M'ds}{I}$ by means of Equations 194, 195, and 196.
6. Figure the value of H_a in terms of H_o, the value of y_o, and the value of M_a in terms of H_a and H_o, using Equations 205, 206, and 207.
7. Figure the values of H_t and M_t, for both rise and fall of temperature, by means of Equations 212 and 213.
8. As a check, compare the values of H_a, M_a, H_t, and M_t just found with those obtained in the preliminary design. The agreement should be close.
9. Calculate the values of M', $\frac{M'ds}{I}$, $\frac{M'yds}{I}$, and $\frac{M'xds}{I}$ for unit loads at each load point, as indicated in Tables 37*d*, 37*e*, 37*f*, and 37*g*, and form the summations for the last three quantities.
10. Calculate the values of H_o, V_o, and M_o for unit loads at each load point by means of the equations written out under Item 5 of this list, and from them calculate the values of V and M (for the said unit loads) at each section which may need testing, using Equations 197, 198, 199, and 200; and record the results as indicated in Table 37*h*.

11. Plot influence lines for the moments at the various sections, as indicated in Fig. 37*jj*. Any irregularities in any of the curves will indicate errors in the calculation work. The apices of the various curves should also lie on a smooth curve, as shown by the dotted line in the same figure. If the loading is distributed rather than concentrated, the curves are then to be utilized to find the moment coefficients for uniform loads. These coefficients differ from the corresponding ones for concentrated loads on account of the fact that the influence lines are curved rather than straight between load points. It will be sufficiently accurate to assume that the two kinds of coefficients are equal except in the case of the coefficient for the moment produced by a load at the point where it acts, and in the case of the coefficients for the moment at the springing. For the former, the value used should be the average ordinate of the portion of the influence line extending from the point in question halfway to the load point on either side; and for the latter, it will be sufficient to use the coefficients for concentrated loads throughout, and then increase the negative moment by the product of the portion of the load extending from the springing halfway to the next load point into the average ordinate of this same portion of the influence line. Table 37*h* should be extended to include the moment coefficients for uniform loads when necessary.
12. Form the summations of the positive moments and the negative moments, and the algebraic sum of these two sums, for each section which may need testing.
13. Figure, for dead load, the values of H_o, V_o, and M_o, and also the values of V and M for each point which may need testing. The unit loads given in Table 37*h* are to be used for this purpose. Record the results as shown in Tables 37*i* and 37*j*, leaving a space in the second table for writing in the values of T and S for such sections as it is later found necessary to test.
14. Figure the live load concentrations, and then compute the maximum positive and negative live load moments at each section which may need testing, using the unit loads given in Table 37*h* if the concentrations are unequal, and the summations given in this same table if the concentrations are all equal. Record the results as shown in Tables 37*k* and 37*l*, leaving space for writing in the values of H and T for such sections as it is later found necessary to test.
15. Calculate H_a and M_a from the equations obtained under Item 6, assuming the arch to be loaded with dead load and one-half live load over its entire length, and then compute the moments due to arch shortening at each section which may need testing, using for this purpose Equation 208. Record the results as

shown in Table 37*m*, leaving space for writing in afterward the values of *T* and *S* for such sections as it may later be found necessary to test.

16. Calculate the moments caused by temperature changes at each section which may need testing, using Equation 214 and the values of H_t found under Item 7. Record the results as shown in Tables 37*n* and 37*o*, leaving space for writing in the values of *T* and *S* for such sections as it may later be found necessary to test.
17. Tabulate the maximum positive and negative moments at each section which may need testing, as shown in Table 37*p*, and form the summations thereof both with and without considering the effects of temperature changes. Leave space for writing in the values of *T* for such sections as it is later found necessary to test.
18. By an examination of the summations of the moments just found, determine which sections must be tested.
19. Figure the dead load thrusts at the critical sections just selected by means of Equation 201 or Equation 202, and record the results in Tables 37*j* and 37*p*.
20. Figure the values of *H* and *V* at the critical sections due to live loads placed in position to produce the maximum moments, using the values for unit loads given in Table 37*h*. Note the values in Tables 37*k* and 37*l*, figure the corresponding values of *T* by means of Equation 201 or 202, and record the results in Tables 37*k*, 37*l*, and 37*p*.
21. Figure the values of *T* at the various critical sections due to arch shortening, using Equation 209, and record the results in Tables 37*m* and 37*p*
22. Figure the values of *T* at the various critical sections due to temperature changes, using Equation 215, and record the results in Tables 37*n*, 37*o*, and 37*p*.
23. Calculate the maximum unit stresses on the concrete and steel at the various critical sections, both with and without considering the effects of temperature.
24. If necessary, figure the thrusts at additional sections, and figure the unit stresses at these sections.
25. Revise the reinforcement to suit the figured stresses, if this be required. It should rarely be necessary to change the sections of the concrete assumed. If, however, it should be found that the sections at first adopted were decidedly incorrect, the entire calculation may have to be done afresh. Generally, however, the approximate formulæ, employed in connection with the design already made, will enable one to effect the revision properly without repeating the complete design; for the amount that the approximate formulæ are in error can easily be determined.

26. Figure the maximum shear at the crown, which is equal to V_o. For the dead load this can be taken from Table 37*j*, and for the live load it can be determined by means of Table 37*h*. The maximum unit shear at the crown will usually be at least almost as large as that at any other section, unless the rib fails to follow the theoretic curve; so that from the unit shear at this point it is possible to tell whether or not it is necessary to figure for shear at other sections. If it be found advisable to make calculations for other sections, the dead-load shears are to be figured by the coefficients given in Table 37*h* (or by simply summing the loads between the crown and the point in question, if the arch be symmetrical) and Equation 203 or Equation 204. These results should be recorded in Table 37*j*, if the sections being tested are given therein. The position of the live load for maximum shear can be determined from Table 37*h*, the ratio $\frac{V}{H_o}$ being less than $\tan\alpha$ for loads producing shear in one direction and greater than $\tan\alpha$ for loads producing shear in the other direction. The values of the live-load shears are to be figured by Table 37*h* and Equation 203 or Equation 204. The shears from arch shortening are to be figured by Equation 210 or Equation 211, and recorded in Table 37*m*, if the sections being tested appear therein. The shears due to temperature changes are to be computed by Equation 216 or Equation 217, and the results recorded in Tables 37*n* and 37*o*, if there happens to be space therein. The total shears are then figured, both with and without considering the effect of temperature changes, and the unit shears are computed. If loading which is really distributed has been assumed concentrated at certain points for calculation purposes, this fact should be considered in computing the shears at various sections.

When choosing the value of N_s under Item 2 above, it will be found most convenient to make it equal to twice the number of divisions of the half rib to be used in performing the integrations. For instance, if it were desired to divide each half of the rib into 10 parts, and N_s were made 10, the values of N at the centres of the various sections would be ½, 1½, 2½, etc.; while by making N_s equal to 20, the said values of N become 1, 3, 5, etc.

In figuring the weight W of the arch rib concentrated at any load point under Item 3, there can be employed the formula,

$$W = 150 \times b \times \frac{l'}{6} (h_1 \sec\alpha_1 + 4h_2 \sec\alpha_2 + h_2 \sec\alpha_2), \qquad \text{[Eq. 218]}$$

in which l' is the distance between load points, h_2 and $\sec\alpha_2$ the values of h and $\sec\alpha$ at the said load point, and h_1, $\sec\alpha_1$, h_3, and $\sec\alpha_3$, their values at the adjacent load points.

Under Item 3, when drawing the figure similar to Fig. 37*ii*, it will frequently be found necessary to plot two or three trial parabolas before deciding whether the value assumed for u is satisfactory. For instance, if in drawing the parabola in Fig. 37*ii* it had been assumed that $P_o = 90,900$, we should have found $P_s = 1.6 \times 90,900 = 145,400$, and the resulting parabola would have fallen entirely below the curve for the actual loads. This would apparently have indicated that the value of u would have to be changed; but by drawing other parabolas with $P_o = 90,000$, 89,000, and 88,000, and $u = 1.6$ in each case, it would have been seen that the value 1.6 was satisfactory.

The magnitude of the dead-load moments produced by a lack of agreement between the parabola and the actual load line can be judged from Fig. 37*ii*. For instance, in the problem illustrated the load P_0 is 2,500 pounds greater than the ordinate to the parabola, and the load P_{12} is about 1,000 pounds less than the ordinate to the said parabola; while the loads at the other points are about the same as the ordinates thereto. By turning to Fig. 37*jj*, the curves of which are similar to those for any other arch, it will be seen that the addition of load at Point 0 and the reduction of load at Point 12 will increase the positive moment near the crown and the negative moment near Point 12. The approximate values of these variations for any particular rib can be computed by assuming that the moment coefficients for the said rib are equal to those in Fig. 37*jj*, multiplied by the ratio of the span length of the rib under consideration to 93.3—the span length of the rib illustrated. Ordinarily, the values of the moments due to this cause will be inappreciable. Should it appear likely in any particular rib that the difference will be important at any section, this section should be investigated.

When figuring the values of H_o, V_o, and M_o for unit loads under Item 10, it is to be noted that M'_l and the summations in which it appears are zero for a load on the right half of the rib, and that M'_r and the summations in which it appears are zero for a load on the left half of the rib.

Under Item 10 it is advisable to be able to determine correctly the sections which need testing, in order to avoid a large waste of time. These critical sections can usually be determined from the preliminary design. For a flat arch, figuring sections at the crown and springing will sometimes be sufficient. It will usually be advisable, however, to test the section at a point or two in the haunch near the crown; for even if it is quite certain that the stresses there are not so high as those at the crown, it is desirable to know whether all of the reinforcement used at the crown is needed in the haunch. It will probably likewise be necessary to consider a point near the springing, in order to determine where a portion of the heavy reinforcement required at the said springing may be stopped off. The approximate methods previously given will generally show whether the figuring of these additional sections is advisable. For an arch of high rise, it will be necessary to test sections at several points, as high stresses are certain to exist throughout the rib. The approximate method will be found of value in locating these critical sections. In general, sections near the crown should be located at load points, where there is an excess of positive moments both from the concentration of load and from arch shortening; sections in the haunches should be taken at load points, where there is an excess of positive moments from the concentration of load, and little stress from arch shortening; and sections near the springing should be taken elsewhere than at load points, in order that the excess of negative moment due to arch shortening may not be partially overcome by the excess of positive moment due to concentration of load. As previously mentioned, it may be necessary to figure certain sections because of irregularities in the loading; and if the centre line of the rib is not laid out to follow the funicular polygon of the loads, the testing of additional sections will frequently be necessary.

In the example illustrated, moment coefficients have been determined for a great many unnecessary points, in order that they may be used for reference.

In figuring the dead-load moments at the load points of an open-spandrel arch (under Item 13), allowance should be made for the fact that the weight of the arch rib itself is not concentrated. This can be done most easily by first figuring the moment as though the entire load were concentrated, and then subtracting (algebraically) from the result the product of the weight of the portion of the rib which is assumed to be concentrated at the load point in question by the approximate difference between the coefficients for concentrated and uniform loads, found as explained under Item 11. A similar correction is to be made in the moment at the springing, the moment figured as for concentrated loads being reduced (algebraically) by the product of the weight of the portion of the rib extending from the said springing half-way to the next load point into the average ordinate of the influence line for this same portion.

When figuring the dead-load values of V under Item 13 for a symmetrical arch, it should be noted that the value thereof at any point is merely the sum of the loads between the crown and the point in question.

Under Items 14 and 20, the conditions of live loading used should strictly be those producing maximum compression in the top and bottom fibres at the various sections, rather than those giving maximum positive and negative moments at the said sections. However, in many instances the two conditions of loading are identical, and the rule as given will rarely lead to errors of consequence. For exact results in any case, there should be loaded not only the load points which cause moment of one sign, but also those producing moment in the opposite direction for which the eccentricity of thrust (found by dividing M by $H_0 \sec \alpha$) is less than one-sixth ($\frac{1}{6}$) of the thickness of the section.

Under Item 15, it is stated that the arch shortening stresses are to be figured for dead load plus half of live load over the entire span. This gives fair average values, and obviates the large amount of work which would be necessary if calculations were made for every condition of loading used. The error involved is negligible.

Under Item 16 it is noted that the moments due to temperature changes are to be computed at all sections which may need testing. For arches of considerable rise, it will be well to figure at first the moments at the crown and springing only. The summaries of the moments at the crown and springing are then made up (Item 17), after which it can be told whether the temperature stresses at other sections need to be figured. The preliminary design will usually indicate whether the effects of temperature changes need to be considered. The calculations called for under Item 22 will, of course, be omitted if it is found that temperature stresses will not affect the design.

Under Item 17 it is unnecessary, in the case of flat arches, to form the summations in which the effect of temperature is not considered, as the temperature stresses are certain to affect the sections.

In order to illustrate the method of design outlined above, there will now be given the complete calculations for the arch rib which has previously been designed in this chapter by the approximate method. The computations for this rib were made in the author's office before the approximate formulæ for design had been worked out, which explains why the form of rib assumed is not that which was shown by the preliminary design to be the best.

The approximate design of this rib assumed that its centre line would be laid out to suit dead plus half live load. If it had been laid out to suit the dead load only, the positive live-load moment at the crown would have been 0.0048 wl^2 rather than 0.0043 wl^2, the moments at the haunch point would have changed but little, and the negative live-load moment at the springing would have been 0.0204 wl^2 instead of 0.0226 wl^2. The sections at the crown and springing will now be tested, assuming the centre line of the rib to be laid out for dead load only, and that the thickness varies in accordance with Equation 178. The figures when temperature change is not considered will not be given.

Section at crown.

Thrust—as before	+ 327,000 lbs.
Moment	
As before	+ 263,000 ft.-lbs.
Add $(0.0048 - 0.0043)\ 2{,}120 \times 93.3^2$	+ 9,000 "
Total	+ 272,000 ft.-lbs.
	or +3,260,000 in.-lbs.
e . . . . 3,260,000 ÷ 327,000	10.0″
Dimensions $h \times b$	24″ × 52″ = 1248 sq. in.
$\frac{h}{e}$	2.4

$R/f_c = \dfrac{3{,}260{,}000}{52\times 24^2\times 780}$ 0.139

$\dfrac{d'}{h} = \dfrac{2.5}{24}$ 0.1

p per face (Fig. 37q) 0.62%

This is satisfactory.

Section at springing.

Thrust—as before .... $+$ 446,000 lbs.

Moment

As before .... $-$ 997,000 ft.-lbs.

Deduct $(0.0226 - 0.0204)2120 \times 93.3^2$ .... 40,000 "

Total .... $-$ 957,000 ft.-lbs.

or $-$ 11,500,000 in.-lbs.

e....11,500,000 ÷ 446,000 .... 25.8″

Dimensions $h \times b$ .... 44″ × 52″ = 2290 sq. in.

$\dfrac{h}{e}$ 1.7

$R/f_c = \dfrac{11{,}500{,}000}{52\times 44^2\times 780}$ 0.147

$\dfrac{d'}{h} = \dfrac{2.5}{44}$ 0.06

p per face (Fig. 37q) 0.56%

This is satisfactory.

Evidently in this case it makes little difference whether the rib is laid out for dead load or for dead load plus half live load. Since the line of pressure passes outside of the springing section in either case, it will be best to reduce the moment at this point as much as possible. The rib will therefore be laid out for dead load only.

We shall assume the following values of r, l, b, h_o, and h_s:

$r = 18.5'$,
$l = 93.3'$,
$b = 4'\,6''$ gross,
$= 4'\,4''$ for figuring weights (allowing for paneling),
$= 4'\,5''$ for figuring moment of inertia (allowing for paneling),
$h_o = 2'\,0'' = 24''$,
$h_s = 3'\,7.7'' = 43.7''$, the theoretic springing being taken some distance inside of the abutment, and the thickness at the face of the abutment being made $3'\,6''$.

The floor-system of this arch will be considered to be divided into 10 panels each 9.33′ long, and to consist of a slab resting on cantilever beams and cross-girders, which are carried on spandrel columns resting on the arch ribs. It will be assumed that this portion of the structure has already been designed, so that the dead load thereof can be readily calculated.

The values of p_o and p_s will first be figured, for dead load only.

p_o.			
Handrails,			
2	× 180 lbs.	= 360	lbs. per lin. ft.
Fascia,			
$2 \times 1.7' \times 2'$	× 150 lbs.	1,020	“ “
Slab,			
$30' \times 0.83'$	× 150 lbs.	3,750	“ “
Cantilever beams,			
$2 \times \frac{1}{2}(1.0' + 2.4') \times 5.4' \div 9.33'$	× 150 lbs.	300	“ “
Cross-girders,			
$17' \times 1' \times 3.5' \div 9.33'$	× 150 lbs.	960	“ “
Spandrel columns,			
$2 \times 3' \times 1.25' \times 3' \div 9.33'$	× 150 lbs.	360	“ “
Arch ribs,			
$2 \times 2' \times 4.33'$	× 150 lbs.	2,600	“ “
Total		9,350	lbs. per lin. ft.
p_s (Theoretic).			
Handrails, fascia, and cantilever beams,			
360 + 1020 + 300		1,680	per lin. ft.
Slab,			
$30' \times 0.86'$	× 150 lbs.	3,870	“ “
Cross-girders,			
$17' \times 1' \times 3.1' \div 9.33'$	× 150 lbs.	850	“ “
Spandrel columns,			
$2 \times 3' \times 1.25' \times 20' \div 9.33'$ ..	× 150 lbs.	2,420	“ “
Arch ribs,			
$2 \times 3.64' \times 4.33' \times 1.3$	× 150 lbs.	6,160	“ “
Total		14,980	per lin. ft.

$$\therefore u = \frac{14{,}980}{9{,}350} = 1.6.$$

The horizontal projection of one-half of the rib will now be divided into twenty equal parts, the point at the crown being numbered 0, and the one at the springing, 20. The loads will be considered to be concentrated at the spandrel columns, which are located at points 0, 4, 8, 12, and 16. For performing the approximate integrations, ten divisions will be used, the centres of these divisions being at the points numbered 1, 3, 5, 7, 9, 11, 13, 15, 17, and 19.

The equations for y, $\frac{dy}{dx}$, and h are now to be written out, using the forms given in Equations 176, 177, and 180, since N_s equals 20. We then have:

$$y = \frac{18.5}{(1.6+5)}\left(\frac{3N^2}{200} + \frac{0.6\,N^4}{16\times 10^4}\right) = 0.04205N^2 + 0.1051\,\frac{N^4}{10^4},$$

$$\frac{dy}{dx} = \frac{18.5}{93.3 \times 6.6}\left(1.2N + \frac{0.6\,N^3}{10^3}\right) = 0.03604N + 0.01802\,\frac{N^3}{10^3},$$

$$\tan\beta = 0.865,$$
$$\sec\beta = 1.322,$$

and $$h = 24'' + \frac{(43.7-24)N\sec\alpha}{20\times 1.322} = 24'' + 0.745N\sec\alpha.$$

The dead-load concentrations at the various load points are next to be figured. As the first step, Table 37*a* is worked out. The values of the various loads are then computed and recorded as follows:

TABLE 37*a*

DATA FOR FIGURING DEAD-LOAD CONCENTRATIONS

N	x	y	dy/dx	$\sec\alpha$	h	$h\sec\alpha$ inches	$h\sec\alpha$ feet
0	0.00′	0.00′	0.000	1.000	24.00″	24.0″	2.00′
4	9.33′	0.68′	0.145	1.011	27.02″	27.4″	2.28′
8	18.67′	2.73′	0.298	1.043	30.22″	31.5″	2.63′
12	28.00′	6.28′	0.464	1.102	33.86″	37.3″	3.11′
16	37.33′	11.46′	0.650	1.193	38.22″	45.6″	3.80′
20	46.67′	18.50′	0.865	1.322	43.70″	57.8″	4.82′

Dead-Load Concentrations.

Point 0.

Handrails and fascia,
1380 × 9.33′............................ 12,900 lbs.
Slab,
3750 × 9.33′ .... 35,000 "
Cantilever beams,
2 × ½(1.1′ + 2.6′) × 5.0′ × 150.......... 2,800 "
Cross-girders,
15.5′ × 4.9′ × 1′ × 150.................. 11,400 "
Spandrel columns,
2 × 3′ × 1.25′ × 3′ × 150 .... 3,400 "
Arch ribs,
2 × 9.33′ × 4.33′ (4 × 2.00′ + 2 × 2.28′) ⅙ ×
150.................................... 25,400 "
Total.................................. 90,900 lbs.
or 45,500 lbs. per rib.

Point 4.

Handrails, fascia, slab, and cantilever beams,
12,900 + 35,000 + 2,800.................. 50,700 lbs.

Cross-girders,
16′ × 3.3′ × 1′ × 150.................... 7,900 "

Spandrel columns,	
$2 \times 3' \times 1.25' \times 3.5' \times 150$	4,000 lbs.
Arch ribs,	
$2 \times 9.33' \times 4.33'(2.00' + 4 \times 2.28' + 2.63')$ $\frac{1}{6} \times 150$	27,900 "
Total	90,500 lbs.
	or 45,300 lbs. per rib.

Point 8.

Handrails, fascia, slab, cantilever beams, and cross-girders, 50,700 + 7,900	58,600 lbs.
Spandrel columns,	
$2 \times 3' \times 1.25' \times 5.3' \times 150$	6,000 "
Arch ribs,	
$2 \times 9.33' \times 4.33'(2.28' + 4 \times 2.63' + 3.11')$ $\frac{1}{6} \times 150$	32,300 "
Total	96,900 lbs.
	or 48,500 lbs. per rib.

Point 12.

Handrails, fascia, slab, cantilever beams, and cross-girders	58,600 lbs.
Spandrel columns,	
$2 \times 3' \times 1.25' \times 8.5' \times 150$	9,600 "
Arch ribs,	
$2 \times 9.33' \times 4.33' (2.63' + 4 \times 3.11' + 3.80)$ $\frac{1}{6} \times 150$	38,300 "
Total	106,500 lbs.
	or 53,300 lbs. per rib.

Point 16.

Handrails, fascia, cantilever beams, and cross-girders, 12,900 + 2,800 + 7,900	23,600 lbs.
Slab, $30(0.83' \times 5.17' + 0.92' \times 4.17') \times 150$	36,300 "
Spandrel cols., $2 \times 3' \times 1.25' \times 13.3' \times 150$	14,900 "
Arch ribs, $2 \times 9.33' \times 4.33' (3.11' + 4 \times 3.80' + 4.81')$ $\frac{1}{6} \times 150$	46,800 "
	121,600 lbs.
	or 60,800 lbs. per rib.

Point 20 (theoretic).

Handrails, fascia, slab, cantilever beams, and cross-girders, 23,600 + 36,300	59,900 lbs.
Spandrel cols., $2 \times 3' \times 1.25' \times 19.6' \times 150$	22,100 "
Arch ribs, $2 \times 9.33' \times 4.33' \times 4.88'$ (average) $\times 150$	59,300 "
	141,300 lbs.
	or 70,700 lbs. per rib.

The half-span of the arch is now drawn to any desired scale (see Fig. 37*ii*), the positions of the load points marked thereon, and the load at each point laid out to any convenient scale. The value of u previously assumed was 1.6. If P_s is 141,300, P_0 should evidently be $\frac{141,300}{1.6}$, or 88,400. A parabola is now drawn through the points $P_0 = 88,400$ and $P_s = 141,300$. The ordinates to this parabola are found to coincide so closely with the computed loads at the various points that no change in the value of u is required.

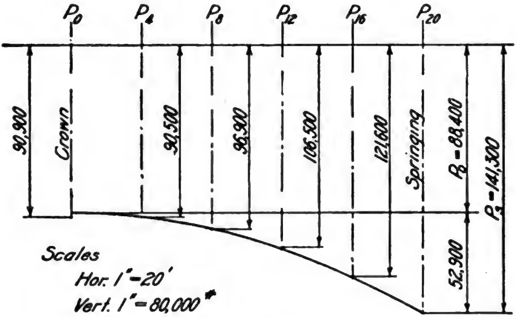


FIG. 37*ii*. Plot of Loads on Arch Rib.

We now proceed to fill out Table 37*b*. The column headed $h - 2d'$ gives the distances between the top and bottom steel, which is assumed to lie 2½" from each face of the rib. The area of the steel in each face will be assumed as 0.5 per cent of the area of the concrete at the crown from Point 0 to Point 16, and as 0.5 per cent of the area of the concrete at the edge of the abutment for the remainder of the rib. The values of A and I will be computed by the following formulæ:

Points 0 to 16,

$$\begin{aligned} A &= 4.42\,h + 2 \times 4.42\,(n - 1) \times 0.01 \\ &= 4.42\,h + 1.24 \\ I &= \frac{4.42\,h^3}{12} + \frac{1.24\,(h - 2d')^2}{4} \\ &= 0.368\,h^3 + 0.31\,(h - 2d')^2. \end{aligned}$$

Points 17 to 20,

$$\begin{aligned} A &= 4.42\,h + 3.5 \times 4.42\,(n - 1) \times 0.01 \\ &= 4.42\,h + 2.17 \\ I &= 0.368\,h^3 + 0.54\,(h - 2d')^2. \end{aligned}$$

Table 37*c* is next filled out. The quantity ds is equal to 4.67 sec α. A check on the values of $\frac{yds}{I}$, $\frac{y^2\,ds}{I}$, $\frac{x\,ds}{I}$, and $\frac{x^2\,ds}{I}$ can be obtained by first figuring $\frac{y^2\,ds}{I}$ and $\frac{x^2\,ds}{I}$ by multiplying $\frac{y\,ds}{I}$ and $\frac{y\,ds}{I}$ by y and x, respectively, and then computing them by multiplying $\frac{ds}{I}$ by x^2 and y^2.

TABLE 37*b*

DIMENSIONS, AREAS, AND MOMENTS OF INERTIA

N	x	y	dy/dx	sec α	h inches	h-$2d'$ inches	h feet	h-$2d'$ feet
0	0.00	0.00	0.000	1.000	24.0	19.0	2.00	1.58
1	2.33	0.04	0.036	1.001	24.8	19.8	2.06	1.65
3	7.00	0.38	0.109	1.006	26.3	21.3	2.19	1.78
5	11.67	1.06	0.183	1.017	27.8	22.8	2.32	1.90
7	16.33	2.09	0.259	1.033	29.4	24.4	2.45	2.03
9	21.00	3.48	0.338	1.056	31.1	26.1	2.59	2.17
11	25.67	5.24	0.420	1.085	32.9	27.9	2.74	2.33
13	30.33	7.41	0.508	1.122	34.9	29.9	2.91	2.49
15	35.00	9.99	0.601	1.167	37.1	32.1	3.09	2.67
17	39.67	13.03	0.701	1.221	39.5	34.5	3.29	2.88
19	44.33	16.55	0.808	1.286	42.2	37.2	3.52	3.10
20	46.67	18.50	0.865	1.322	43.7	38.7	3.64	3.22

N	$A_{conc.}$	A_{steel}	A	$I_{conc.}$	I_{steel}	I
0	8.84	1.24	10.08	2.94	0.78	3.72
1	9.11	1.24	10.35	3.23	0.84	4.07
3	9.69	1.24	10.93	3.85	0.97	4.82
5	10.26	1.24	11.50	4.57	1.12	5.69
7	10.84	1.24	12.08	5.40	1.28	6.68
9	11.46	1.24	12.70	6.40	1.47	7.87
11	12.12	1.24	13.36	7.60	1.68	9.28
13	12.88	1.24	14.12	9.05	1.92	10.97
15	13.68	1.24	14.92	10.85	2.22	13.07
17	14.56	2.17	16.73	13.13	4.46	17.59
19	15.58	2.17	17.75	16.05	5.20	21.25
20	16.11	2.17	18.28	17.78	5.61	23.39

TABLE 37*c*

VALUES OF ds, $\frac{ds}{I}$, $\frac{y\,ds}{I}$, $\frac{y^2\,ds}{I}$, $\frac{x\,ds}{I}$, $\frac{x^2\,ds}{I}$, AND $\frac{\sec\alpha\,ds}{A}$.

N	ds	$\frac{ds}{I}$	$\frac{y\,ds}{I}$	$\frac{y^2\,ds}{I}$	$\frac{x\,ds}{I}$	$\frac{x^2\,ds}{I}$	$\frac{\sec\alpha\,ds}{A}$
1	4.67	1.147	0.05	0.0	2.68	6	0.452
3	4.69	0.972	0.37	0.1	6.80	48	0.431
5	4.75	0.834	0.88	0.9	9.73	114	0.420
7	8.82	0.722	1.51	3.2	11.81	193	0.413
9	4.93	0.626	2.18	7.6	13.15	276	0.410
11	5.06	0.545	2.86	15.0	14.01	360	0.411
13	5.24	0.477	3.54	26.2	14.48	439	0.417
15	5.45	0.417	4.17	41.7	14.61	511	0.426
17	5.70	0.324	4.22	55.1	12.86	511	0.417
19	6.00	0.282	4.66	77.1	12.51	555	0.434
Σ	51.31	6.346	24.44	226.9	112.64	3013	4.231

The values of H_o, V_o, and M_o, in terms of the quantities $\Sigma M'ds/I$, $\Sigma M'y\,ds/I$, and $\Sigma M'x\,ds/I$, are next found as follows:

$$H_o = \frac{6.346\,\Sigma M'y\,ds/I - 24.44\,\Sigma M'ds/I}{2\,(24.44^2 - 6.346 \times 226.9)} = \frac{-\,6.346\,\Sigma M'y\,ds/I + 24.44\,\Sigma M'ds/I}{2 \times 842.6}$$

$$= -\,0.003766\,\Sigma M'y\,ds/I + 0.01450\,\Sigma M'ds/I.$$

$$V_o = \frac{1}{6026}\,(\Sigma M'_l\,x\,ds/I - \Sigma M'_r\,x\,ds/I).$$

$$M_o = -\,\frac{2 \times 24.44\,H_o + \Sigma M'ds/I}{2 \times 6.346} = -\,3.851\,H_o - 0.0788\,\Sigma M'ds/I.$$

$$= +\,0.01450\,\Sigma M'y\,ds/I - 0.1346\,\Sigma M'ds/I.$$

The values of H_a in terms of H_o, that of y_o, and that of M_a in terms of H_a and H_o, are next computed thus:

$$H_a = -\,\frac{4.231 \times 6.346\,H_o}{1.322 \times 842.6 + 4.231 \times 6.346} = -\,\frac{26.9\,H_o}{1141} = -\,0.0236\,H_o.$$

$$y_o = \frac{24.44}{6.346} = 3.851'.$$

$$M_a = -\,3.851\,H_a = 0.0909\,H_o.$$

The values of H_a and y_o check very well with those obtained by the approximate method, as can be seen by comparison.

The values of H_t and M_t for a 50° fall of temperature are:

$$H_t = -\,\frac{288{,}000{,}000 \times 0.000{,}006 \times 50 \times 93.3 \times 6.346 \times 1.322}{2 \times 1141} = -\,29{,}700 \text{ lbs.}$$

$M_t = 29{,}700 \times 3.851 = +\,114{,}000$ ft.-lbs.

The values of H_t and M_t for a 30° rise of temperature are:

$H_t = 29{,}700 \times \frac{3}{5} = +\,17{,}800$ lbs.

$M_t = -\,114{,}000 \times \frac{3}{5} = -\,68{,}000$ ft.-lbs.

The values of H_t and M_t also check closely with those obtained by the approximate formulæ.

The values of M', $\frac{M'ds}{I}$, $\frac{M'y\,ds}{I}$, and $\frac{M'x\,ds}{I}$ for unit loads at each of the load points are now figured, and the results recorded in Tables 37*d*, 37*e*, 37*f*, and 37*g*; and the summations are found for the three latter quantities.

The values of H_o, V_o, and M_o for unit loads at each load point are now figured by means of the equations determined previously, and the results recorded in Table 37*h*. The load at the crown is treated as being on the right half of the rib.

TABLE 37*d*

VALUES OF M' FOR UNIT LOADS

N	POINT LOADED				
	0	4	8	12	16
1	− 2.33	....	....	....	......
3	− 7.00	....	....	....	......
5	−11.67	− 2.33	....	....	......
7	−16.33	− 7.00	....	....	......
9	−21.00	−11.67	− 2.33	....	......
11	−25.67	−16.33	− 7.00	....	......
13	−30.33	−21.00	−11.67	− 2.33	......
15	−35.00	−25.67	−16.33	− 7.00	......
17	−39.67	−30.33	−21.00	−11.67	−2.33
19	−44.33	−35.00	−25.67	−16.33	−7.00

TABLE 37*e*

VALUES OF $\frac{M'\,ds}{I}$ FOR UNIT LOADS

N	POINT LOADED				
	0	4	8	12	16
1	− 2.68	....	....	....	......
3	− 6.80	....	....	....	......
5	− 9.73	− 1.95	....	....	......
7	−11.81	− 5.05	....	....	......
9	−13.15	− 7.30	− 1.46	....	......
11	−14.01	− 8.91	− 3.82	....	......
13	−14.48	−10.03	− 5.57	− 1.11	......
15	−14.61	−10.71	− 6.81	− 2.92	......
17	−12.86	− 9.83	− 6.81	− 3.78	−0.76
19	−12.51	− 9.87	− 7.24	− 4.61	−1.97
Σ	−112.64	−63.65	−31.71	−12.42	−2.73

TABLE 37*f*

VALUES OF $\frac{M'y\,ds}{I}$ FOR UNIT LOADS

N	POINT LOADED				
	0	4	8	12	16
1	− 0.1	....	....	....	......
3	− 2.6	....	....	....	......
5	− 10.3	− 2.1	....	....	......
7	− 24.7	− 10.6	....	....	......
9	− 45.8	− 25.4	− 5.1	....	......
11	− 73.4	− 46.7	− 20.0	....	......
13	−107.4	− 74.3	− 41.3	− 8.2	......
15	−146.0	−107.0	− 68.2	− 29.2	......
17	−167.5	−128.0	− 88.6	− 49.2	− 9.9
19	−206.9	−163.2	−119.7	− 76.2	−32.6
Σ	−784.7	−557.3	−342.9	−162.8	− 42.5

TABLE 37*g*

VALUES OF $\frac{M' x\, ds}{I}$ FOR UNIT LOADS

N	Point Loaded				
	0	4	8	12	16
1	− 6	......	......	.....	.....
3	− 48	......	......	.....	.....
5	− 114	− 23	......	.....	.....
7	− 193	− 83	......	.....	.....
9	− 276	− 153	− 31	.....	.....
11	− 360	− 229	− 98	.....	.....
13	− 439	− 304	− 169	− 34	.....
15	− 511	− 375	− 239	−102	.....
17	− 511	− 390	− 270	−150	− 30
19	− 555	− 438	− 321	−204	− 87
Σ	−3013	−1995	−1128	−490	−117

TABLE 37*h*

VALUES OF *H*, *V*, AND *M* AT VARIOUS SECTIONS IN THE LEFT HALF OF RIB, FOR UNIT LOADS AT EACH LOAD POINT

(Load at crown treated as being on right half of rib.)

Point Loaded	Crown			Point 1		Point 3	
	H_O	V_O	M_O	V_1	M_1	V_3	M_3
16L	+0.120	−0.019	−0.23	−0.019	−0.18	−0.019	−0.05
12L	+0.434	−0.081	−0.69	−0.081	−0.48	−0.081	+0.04
8L	+0.831	−0.187	−0.70	−0.187	−0.23	−0.187	+0.93
4L	+1.175	−0.331	+0.49	−0.331	+1.31	−0.331	+3.26
0	+1.322	+0.500	+3.79	+0.500	+2.67	+0.500	+0.79
4R	+1.175	+0.331	+0.49	+0.331	−0.24	+0.331	−1.38
8R	+0.831	+0.187	−0.70	+0.187	−1.10	+0.187	−1.70
12R	+0.434	+0.081	−0.69	+0.081	−0.86	+0.081	−1.09
16R	+0.120	+0.019	−0.23	+0.019	−0.27	+0.019	−0.32
Σ Positive *M*	....	....	+4.77	....	+3.98	....	+5.02
Σ Negative *M*	....	....	−3.24	....	−3.36	....	−4.54
Σ *M*	6.442	....	+1.53	....	+0.62	....	+0.48

Point Loaded	Point 4		Point 5		Point 7		Point 8	
	V_4	M_4	V_5	M_5	V_7	M_7	V_8	M_8
16L	−0.019	+0.03	−0.019	+0.12	−0.019	+0.33	−0.019	+0.46
12L	−0.081	+0.36	−0.081	+0.72	−0.081	+1.54	−0.081	+2.01
8L	−0.187	+1.62	−0.187	+2.37	−0.187	+4.10	+0.813	+5.07
4L	+0.669	+4.38	+0.669	+3.27	+0.669	+1.36	+0.669	+0.55
0	+0.500	+0.02	+0.500	−0.64	+0.500	−1.61	+0.500	−1.93
4R	+0.331	−1.80	+0.331	−2.13	+0.331	−2.46	+0.331	−2.48
8R	+0.187	−1.89	+0.187	−2.00	+0.187	−2.02	+0.187	−1.93
12R	+0.081	−1.15	+0.081	−1.17	+0.081	−1.11	+0.081	−1.02
16R	+0.019	−0.32	+0.019	−0.33	+0.019	−0.29	+0.019	−0.25
Σ Pos. *M*	....	+6.41	....	+6.48	....	+7.33	....	+8.09
Σ Neg. *M*	....	−5.16	....	−6.27	....	−7.49	....	−7.61
Σ *M*	....	+1.25	....	+0.21	....	−0.16	....	+0.48

TABLE 37*h* (*Continued*)

Point Loaded	Point 9		Point 11		Point 13		Point 15	
	V_9	M_9	V_{11}	M_{11}	V_{13}	M_{13}	V_{15}	M_{15}
16L	−0.019	+0.59	−0.019	+0.89	−0.019	+1.24	−0.019	+1.63
12L	−0.081	+2.52	−0.081	+3.66	+0.919	+2.64	+0.919	−0.53
8L	+0.813	+3.79	+0.813	+1.46	+0.813	−0.53	+0.813	−2.17
4L	+0.669	−0.13	+0.669	−1.18	+0.669	−1.75	+0.669	−1.84
0	+0.500	−2.10	+0.500	−2.11	+0.500	−1.57	+0.500	−0.49
4R	+0.331	−2.38	+0.331	−1.86	+0.331	−0.85	+0.331	+0.64
8R	+0.187	−1.74	+0.187	−1.15	+0.187	−0.22	+0.187	+1.05
12R	+0.081	−0.88	+0.081	−0.50	+0.081	+0.06	+0.081	+0.80
16R	+0.019	−0.21	+0.019	−0.09	+0.019	+0.08	+0.019	+0.30
Σ Pos. *M*	....	+6.90	....	+6.01	....	+4.02	....	+4.42
Σ Neg. *M*	....	−7.44	....	−6.89	....	−4.92	....	−5.03
Σ *M*	....	−0.54	....	−0.88	....	−0.90	....	−0.61

Point Loaded	Point 17		Point 19		Point 20	
	V_{17}	M_{17}	V_{19}	M_{19}	V_{20}	M_{20}
16L	+0.981	− 0.25	+0.981	− 4.40	+0.981	− 6.45
12L	+0.919	− 3.49	+0.919	− 6.25	+0.919	− 7.54
8L	+0.813	− 3.44	+0.813	− 4.32	+0.813	− 4.59
4L	+0.669	− 1.39	+0.669	− 0.38	+0.669	+ 0.35
0	+0.500	+ 1.20	+0.500	+ 3.51	+0.500	+ 4.92
4R	+0.331	+ 2.67	+0.331	+ 5.26	+0.331	+ 6.78
8R	+0.187	+ 2.70	+0.187	+ 4.76	+0.187	+ 5.94
12R	+0.081	+ 1.75	+0.081	+ 2.90	+0.081	+ 3.56
16R	+0.019	+ 0.58	+0.019	+ 0.91	+0.019	+ 1.10
Σ Pos. *M*	....	+ 8.90	....	+17.34	....	+22.65
Σ Neg. *M*	....	− 8.57	+3.382	−15.35	....	−18.58
Σ *M*	....	+ 0.33	....	+ 1.99	....	+ 4.07

It is next necessary to determine the sections which may need testing. In laying out this rib, the springing (Point 20) was assumed some distance within the abutment, Point 19 falling about at the face thereof; so that the section at Point 20 will, therefore, not need testing. The crown and Point 19 must, of course, be figured; and as the arch is only fairly flat $\left(\frac{r}{l} = \frac{1}{5}\right)$, the load point nearest to the crown—Point 4—must be tested. The next load point—Point 8—is not very likely to need testing. By turning back to the preliminary design, we note that the section at Point 4 is not so highly stressed as that at the crown, and by making similar figures for Point 8 it is evident the stresses here will be still less. It will be advisable to test both of these sections, however. One point near Point 19—say that at Point 17—will probably be sufficient. We, therefore, need to figure the moments for sections at the crown, Point 4, Point 8, Point 17, and Point 19. As has been previously stated, the moments at several other points will be figured, in order to give a complete set of values for reference. These additional points will be Points 1, 3, 5, 7, 9, 11, 13, 15, and 20.

The values of M and V at the various sections in the left half of the rib due to unit loads at the various load points are now figured by means of Equations 197 and 199. V' for any section is equal to zero for a load on the right half of the rib or between the left springing and the section in question, and to unity for a load between the crown and the said section. The results are recorded in Table 37*h*. The calculations for the values of M can be conveniently arranged to reduce the labor to a minimum, as shown herewith for a load at Point 16 on the left half of the

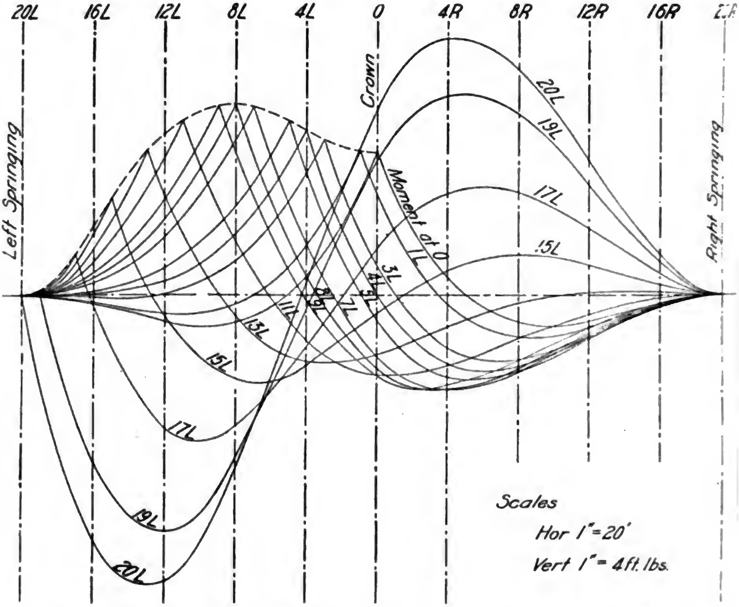


FIG. 37*jj*. Influence Lines for Moments at Various Sections in the Left-Half of Rib, for Unit Loads at Each Load Point.

Section	20*l*	19*l*	17*l*	15*l*	13*l*	11*l*	9*l*	8*l*	7*l*	5*l*	4*l*	3*l*	1*l*
M'	−9.33	−7.00	−2.33	0	0	0	0	0	0	0	0	0	0
M_o	−0.23	−0.23	−0.23	−0.23	−0.23	−0.23	−0.23	−0.23	−0.23	−0.23	−0.23	−0.23	−0.23
H_oy	+2.22	+1.99	+1.56	+1.20	+0.89	+0.63	+0.42	+0.33	+0.25	+0.13	+0.08	+0.05	+0.01
$-V_ox$	+0.89	+0.84	+0.75	+0.66	+0.58	+0.49	+0.40	+0.36	+0.31	+0.22	+0.18	+0.13	+0.04
M	−6.45	−4.40	−0.25	+1.63	+1.24	+0.89	+0.59	+0.46	+0.33	+0.12	+0.03	−0.05	−0.18

rib. Only two settings of the slide rule are required, one for the products $H_o y$, the other for the products $V_o x$.

The influence lines for the moments at the various sections due to the unit loads are then drawn for a check, as shown in Fig. 37*jj*. (When the calculations were first made, a few important errors therein produced irregularities in the curves, and were thus detected.)

The summations of all of the positive moments at each section are then formed, and also those for the negative moments. The algebraic sum of these two summations for each section is also obtained.

The values of H_o, V_o, and M_o for dead load are then computed by means of the values for unit loads given in Table 37*h*, and the results are recorded in Table 37*i*. The value of M_o for the load at Point 0 is

TABLE 37*i*

DEAD-LOAD STRESSES

Point Loaded	Load	Crown—Point 0			Point 1	
		H_o	V_o	M_o	V	M
16 R and L	60,800	+ 14,600	0	− 28,000	0	− 27,000
12 R and L	53,300	+ 46,300	0	− 74,000	0	− 72,000
8 R and L	48,500	+ 80,600	0	− 68,000	0	− 65,000
4 R and L	45,300	+106,600	0	+ 44,000	0	+ 49,000
0	45,500	+ 60,100	+ 22,800	+160,000	+ 22,800	+122,000
Σ	......	+308,200	+ 22,800	+ 34,000	+ 22,800	+ 7,000

Point Loaded	Point 3		Point 4		Point 5	
	V	M	V	M	V	M
16 R and L	0	− 23,000	0	− 18,000	0	− 13,000
12 R and L	0	− 56,000	0	− 42,000	0	− 24,000
8 R and L	0	− 37,000	0	− 13,000	0	+ 18,000
4 R and L	0	+ 85,000	+ 45,300	+103,000	+ 45,300	+ 52,000
0	+ 22,800	+ 36,000	+ 22,800	+ 1,000	+ 22,800	− 29,000
Σ	+ 22,800	+ 5,000	+ 68,100	+ 31,000	+ 68,100	+ 4,000

Point Loaded	Point 7		Point 8		Point 9	
	V	M	V	M	V	M
16 R and L	0	+ 2,000	0	+ 13,000	0	+ 23,000
12 R and L	0	+ 23,000	0	+ 53,000	0	+ 87,000
8 R and L	0	+101,000	+ 48,500	+134,000	+ 48,500	+ 99,000
4 R and L	+ 45,300	− 50,000	+ 45,300	− 87,000	+ 45,300	−114,000
0	+ 22,800	− 73,000	+ 22,800	− 88,000	+ 22,800	− 96,000
Σ	+ 68,100	+ 3,000	+116,600	+ 25,000	+116,600	− 1,000

TABLE 37*i* (*Continued*)

Point Loaded	Point 11		Point 13		Point 15	
	V	*M*	*V*	*M*	*V*	*M*
16 R and L	0	+ 49,000	0	+ 80,000	0	+117,000
12 R and L	0	+169,000	+ 53,300	+144,000	+ 53,300	+ 14,000
8 R and L	+ 48,500	+ 15,000	+ 48,500	− 36,000	+ 48,500	− 54,000
4 R and L	+ 45,300	−138,000	+ 45,300	−118,000	+ 45,300	− 54,000
0	+ 22,800	− 96,000	+ 22,800	− 71,000	+ 22,800	− 22,000
Σ	+116,600	− 1,000	+169,900	− 1,000	+169,900	+ 1,000

Point Loaded	Point 17		Point 19		Point 20	
	V	*M*	*V*	*M*	*V*	*M*
16 R and L	+ 60,800	+ 20,000	+ 60,800	−212,000	+ 60,800	−325,000
12 R and L	+ 53,300	− 93,000	+ 53,300	−179,000	+ 53,300	−212,000
8 R and L	+ 48,500	− 36,000	+ 48,500	+ 21,000	+ 48,500	+ 66,000
4 R and L	+ 45,300	+ 58,000	+ 45,300	+221,000	+ 45,300	+323,000
0	+ 22,800	+ 55,000	+ 22,800	+160,000	+ 22,800	+224,000
Σ	+230,700	+ 4,000	+230,700	+ 11,000	+230,700	+*43,000

M_0 for load at Point 0 = 45,500 × 3.79 − 12,700 × 1.0 = 160,000 ft. lbs.
M_4 for load at Point 4 = 45,300 × 2.58 − 14,000 × 1.0 = 103,000 ft. lbs.
M_8 for load at Point 8 = 48,500 × 3.14 − 16,300 × 1.0 = 134,000 ft. lbs.

*Reduction in M_{20} for weight of arch rib near springing = ½ × 29,700 × 2.2 = 33,000 ft. lbs.

reduced, as noted at the bottom of the table, to allow for the fact that the weight of the arch rib is not concentrated, as is explained under Item 11. The values of *M* at each section are next figured in a similar manner, and the results are tabulated in Table 37*i*. The moment at Point 4 due to the load at that point is reduced as shown at the bottom of the table,

TABLE 37*j*

SUMMARY OF DEAD-LOAD STRESSES

Section	*H*	*V*	*T*	*S*	*M*
0	+308,200	+ 22,800	+308,000	+23,000	+34,000
1	....	....	....	....	+ 7,000
3	....	....	....	....	+ 5,000
4	+308,200	+ 68,100	+315,000	....	+31,000
5	....	....	....	....	+ 4,000
7	....	....	....	....	+ 3,000
8	....	....	....	....	+25,000
9	....	....	....	....	− 1,000
11	....	....	....	....	− 1,000
13	....	....	....	....	− 1,000
15	....	....	....	....	+ 1,000
17	+308,200	+230,700	+384,000	....	+ 4,000
19	+308,200	+230,700	+384,000	....	+11,000
20	....	....	....	....	+43,000

as is also the moment at Point 8; and the moment at the springing is also properly reduced, one-half of 29,700 pounds being the weight of the portion of the arch rib extending from the springing to Point 18. (See Dead-Load Concentrations, Point 20.)

The total dead-load stresses at the various sections are then tabulated in Table 37*j*.

The live-load concentration is next figured. From the preliminary design, we know that the live plus impact load per lineal foot of rib is 2,120 pounds, so that the value of a concentration is evidently 2120 × 9.33′ = 20,000 lbs. approximately.

The maximum positive and maximum negative live-load moments at each section are then figured by means of the summations given at the bottom of Table 37*h*, and the results are tabulated in Tables 37*k* and 37*l*.

TABLE 37*k*

MAXIMUM POSITIVE LIVE-LOAD MOMENTS

Section	Points Loaded	*H*	*V*	*T*	*M*
0	4L– 4R	+73,000	+10,000	+73,000	+ 95,000
1	4L– 0	....	....	....	+ 80,000
3	12L– 0	....	....	....	+100,000
4	16L– 0	+78,000	+18,000	+80,000	+128,000
5	16L– 4L	....	....	....	+130,000
7	16L– 4L	....	....	....	+147,000
8	16L– 4L	....	....	....	+162,000
9	16L– 8L	....	....	....	+138,000
11	16L– 8L	....	....	....	+120,000
13	16L, 12L, 12R, 16R	....	....	....	+ 80,000
15	16L, 4R–16R	....	....	....	+ 88,000
17	0 –16R	....	....	....	+178,000
19	0 –16R	....	....	....	+347,000
20	4L–16R	....	....	....	+453,000

TABLE 37*l*

MAXIMUM NEGATIVE LIVE-LOAD MOMENTS

Section	Points Loaded	*H*	*V*	*T*	*M*
0	16L–8L, 8R–16R	....	....	....	– 65,000
1	16L–8L, 4R–16R	....	....	....	– 67,000
3	16L, 4R–16R	....	....	....	– 91,000
4	4R–16R	....	....	....	–103,000
5	0 –16R	....	....	....	–125,000
7	0 –16R	....	....	....	–150,000
8	0 –16R	....	....	....	–152,000
9	4L–16R	....	....	....	–149,000
11	4L–16R	....	....	....	–138,000
13	8L– 8R	....	....	....	– 98,000
15	12L– 0	....	....	....	–101,000
17	16L– 4L	+51,000	+68,000	+81,000	–171,000
19	16L– 4L	+51,000	+68,000	+82,000	–307,000
20	16L– 8L	....	....	....	–372,000

The value of H_a, the thrust at the crown due to arch shortening, is next figured for dead-plus-half-live load over the entire span. The value of H_o is obtained thus:

Dead load—see Table 37*j*. . . . . + 308,200

Half live load—$\frac{20{,}000}{2} \times 6.442$ (by Table 37*h*). . . . . . + 64,400

H_o = ... + 372,600

The value of H_a is therefore

$$H_a = -\ 0.0236 \times 372{,}600 = -\ 8{,}800 \text{ lbs.}$$

The moments at the various sections due to arch shortening, to rise of temperature, and to fall of temperature are then figured, and the results are tabulated in Tables 37*m*, 37*n*, and 37*o*. *H* is constant throughout each table, being equal to the value at the crown; and *V* is zero.

TABLE 37*m*

STRESSES FROM ARCH SHORTENING

Section	$y - 3.85$	H_a	V	T	S	M
0	− 3.85	− 8,800	0	− 9,000	....	+ 34,000
1	− 3.81	− 8,800	0	....	....	+ 34,000
3	− 3.47	− 8,800	0	....	....	+ 31,000
4	− 3.17	− 8,800	0	− 9,000	....	+ 28,000
5	− 2.79	− 8,800	0	....	....	+ 25,000
7	− 1.76	− 8,800	0	....	....	+ 15,000
8	− 1.12	− 8,800	0	....	....	+ 10,000
9	− 0.37	− 8,800	0	....	....	+ 3,000
11	+ 1.39	− 8,800	0	....	....	− 12,000
13	+ 3.56	− 8,800	0	....	....	− 31,000
15	+ 6.14	− 8,800	0	....	....	− 54,000
17	+ 9.18	− 8,800	0	− 7,000	....	− 81,000
19	+12.70	− 8,800	0	− 7,000	....	−112,000
20	+14.65	− 8,800	0	....	....	−129,000

TABLE 37*n*

STRESSES FROM 30° RISE OF TEMPERATURE

Section	$y - 3.85$	H_t	V	T	S	M
0	− 3.85	+17,800	0	+18,000	....	− 68,000
1	− 3.81	+17,800	0	....	....	− 68,000
3	− 3.47	+17,800	0	....	....	− 62,000
4	− 3.17	+17,800	0	+18,000	....	− 56,000
5	− 2.79	+17,800	0	....	....	− 50,000
7	− 1.76	+17,800	0	....	....	− 31,000
8	− 1.12	+17,800	0	....	....	− 20,000
9	− 0.37	+17,800	0	....	....	− 7,000
11	+ 1.39	+17,800	0	....	....	+ 25,000
13	+ 3.56	+17,800	0	....	....	+ 63,000
15	+ 6.14	+17,800	0	....	....	+109,000
17	+ 9.18	+17,800	0	+15,000	....	+163,000
19	+12.70	+17,800	0	+14,000	....	+226,000
20	+14.65	+17,800	0	....	....	+261,000

TABLE 37*o*

STRESSES FROM 50° FALL OF TEMPERATURE

Section	$y-3.85$	H_t	V	T	S	M
0	− 3.85	−29,700	0	−30,000	....	+114,000
1	− 3.81	−29,700	0	....	....	+113,000
3	− 3.47	−29,700	0	....	....	+103,000
4	− 3.17	−29,700	0	−29,000	....	+ 94,000
5	− 2.79	−29,700	0	....	....	+ 83,000
7	− 1.76	−29,700	0	....	....	+ 52,000
8	− 1.12	−29,700	0	....	....	+ 33,000
9	− 0.37	−29,700	0	....	....	+ 11,000
11	+ 1.39	−29,700	0	....	....	− 41,000
13	+ 3.56	−29,700	0	....	....	−106,000
15	+ 6.14	−29,700	0	....	....	−182,000
17	+ 9.18	−29,700	0	−24,000	....	−273,000
19	+12.70	−29,700	0	−23,000	....	−377,000
20	+14.65	−29,700	0	....	....	−435,000

Table 37*p*, giving the maximum positive and maximum negative moments at the various sections, is next made up. The maximum moments, both with and without the effect due to a change of temperature, are figured for each section. Remembering that Point 19 is at the face of the abutment, it is evident that we need to test the crown and Point 19 only, so far as the concrete sections are concerned; and Points 4 and 17, and, possibly, Points 8 and 15, to determine the reinforcement. The values of the thrusts at the crown and at Points 4, 17, and 19 are, therefore, computed as explained under Items 19, 20, 21, and 22, such blanks being filled out in Tables 37*j* to 37*p*, inclusive, as are found necessary. Evidently we need to figure for maximum positive moments only at the crown and Point 4, and for maximum negative moments only at Points 17 and 19.

The sections at these four points are now tested as follows:

Section at Crown. Dimensions $h \times b$,	Temperature Not Considered	Temperature Considered
24 × 53	1272 sq. in.	1272 sq. in.
Moment...............	+ 163,000 ft. lbs.	+ 277,000 ft. lbs.
Thrust................	+ 372,000 lbs.	+ 342,000 lbs.
e	0.44′ = 5.2″	0.81′ = 9.7″
$\frac{h}{e}$	4.6	2.5
$\frac{R}{f_c}$	$\frac{163{,}000 \times 12}{53 \times 24^2 \times 600} = 0.107$	$\frac{277{,}000 \times 12}{53 \times 24^2 \times 780} = 0.140$
$\frac{d'}{h}$	$\frac{2.5}{24} = 0.1$	0.1
p per face (Fig. 37*q*)..	...	0.64%

This result is satisfactory.

Use 11 1″ round bars in each face, making $p = \frac{11 \times 0.785}{1272} = 0.68\%$.

TABLE 37*p*

MAXIMUM POSITIVE AND NEGATIVE MOMENTS AT VARIOUS SECTIONS

Section	Loading	Maximum Positive Moments		Maximum Negative Moments	
		Thrust	Moment	Thrust	Moment
0	*D*	+308,000	+ 34,000	....	+ 34,000
	L	+ 73,000	+ 95,000	....	− 65,000
	AS	− 9,000	+ 34,000	....	+ 34,000
	T	− 30,000	+114,000	....	− 68,000
	D+*L*+*AS*	+372,000	+163,000	....	+ 3,000
	D+*L*+*AS*+*T*	+342,000	+277,000	....	− 65,000
1	*D*	....	+ 7,000	....	+ 7,000
	L	....	+ 80,000	....	− 67,000
	AS	....	+ 34,000	....	+ 34,000
	T	....	+113,000	....	− 68,000
	D+*L*+*AS*	....	+121,000	....	− 26,000
	D+*L*+*AS*+*T*	....	+234,000	....	− 94,000
3	*D*	....	+ 5,000	....	+ 5,000
	L	....	+100,000	....	− 91,000
	AS	....	+ 31,000	....	+ 31,000
	T	....	+103,000	....	− 62,000
	D+*L*+*AS*	....	+136,000	....	− 55,000
	D+*L*+*AS*+*T*	....	+239,000	....	−117,000
4	*D*	+315,000	+ 31,000	....	+ 31,000
	L	+ 80,000	+128,000	....	−103,000
	AS	− 9,000	+ 28,000	....	+ 28,000
	T	− 29,000	+ 94,000	....	− 56,000
	D+*L*+*AS*	+386,000	+187,000	....	− 44,000
	D+*L*+*AS*+*T*	+357,000	+281,000	....	−100,000
5	*D*	....	+ 4,000	....	+ 4,000
	L	....	+130,000	....	−125,000
	AS	....	+ 25,000	....	+ 25,000
	T	....	+ 83,000	....	− 50,000
	D+*L*+*AS*	....	+159,000	....	− 96,000
	D+*L*+*AS*+*T*	....	+242,000	....	−146,000
7	*D*	....	+ 3,000	....	+ 3,000
	L	....	+147,000	....	−150,000
	AS	....	+ 15,000	....	+ 15,000
	T	....	+ 52,000	....	− 31,000
	D+*L*+*AS*	....	+165,000	....	−132,000
	D+*L*+*AS*+*T*	....	+217,000	....	−163,000
8	*D*	....	+ 25,000	....	+ 25,000
	L	....	+162,000	....	−152,000
	AS	....	+ 10,000	....	+ 10,000
	T	....	+ 33,000	....	− 20,000
	D+*L*+*AS*	....	+197,000	....	−117,000
	D+*L*+*AS*+*T*	....	+230,000	....	−137,000

TABLE 37*p* (*Continued*)

Section	Loading	Maximum Positive Moments		Maximum Negative Moments	
		Thrust	Moment	Thrust	Moment
9	D	....	− 1,000	....	− 1,000
	L	....	+138,000	....	−149,000
	AS	....	+ 3,000	....	+ 3,000
	T	....	+ 11,000	....	− 7,000
	$D+L+AS$	....	+140,000	....	−147,000
	$D+L+AS+T$	....	+151,000	....	−154,000
11	D	....	− 1,000	....	− 1,000
	L	....	+120,000	....	−138,000
	AS	....	− 12,000	....	− 12,000
	T	....	+ 25,000	....	− 41,000
	$D+L+AS$	....	+107,000	....	−151,000
	$D+L+AS+T$	....	+132,000	....	−192,000
13	D	....	− 1,000	....	− 1,000
	L	....	+ 80,000	....	− 98,000
	AS	....	− 31,000	....	− 31,000
	T	....	+ 63,000	....	−106,000
	$D+L+AS$	....	+ 48,000	....	−130,000
	$D+L+AS+T$	....	+111,000	....	−236,000
15	D	....	+ 1,000	....	+ 1,000
	L	....	+ 88,000	....	−101,000
	AS	....	− 54,000	....	− 54,000
	T	....	+109,000	....	−182,000
	$D+L+AS$	....	+ 35,000	....	−154,000
	$D+L+AS+T$	....	+144,000	....	−336,000
17	D	....	+ 4,000	+384,000	+ 4,000
	L	....	+178,000	+ 81,000	−171,000
	AS	....	− 81,000	− 7,000	− 81,000
	T	....	+163,000	− 24,000	−273,000
	$D+L+AS$	....	+101,000	+458,000	−248,000
	$D+L+AS+T$	....	+264,000	+434,000	−521,000
19	D	....	+ 11,000	+384,000	+ 11,000
	L	....	+347,000	+ 82,000	−307,000
	AS	....	−112,000	− 7,000	−112,000
	T	....	+226,000	− 23,000	−377,000
	$D+L+AS$	....	+246,000	+459,000	−408,000
	$D+L+AS+T$	....	+472,000	+436,000	−785,000
20	D	....	+ 43,000	....	+ 43,000
	L	....	+453,000	....	−372,000
	AS	....	−129,000	....	−129,000
	T	....	+261,000	....	−435,000
	$D+L+AS$	....	+367,000	....	−458,000
	$D+L+AS+T$	....	+628,000	....	−893,000

Section at Point 4.	Temperature Not Considered	Temperature Considered
Dimensions $h \times b$, 27 × 53	1431 sq. in.	1431 sq. in.
Moment	+ 187,000 ft. lbs.	+ 281,000 ft. lbs.
Thrust	+ 386,000 lbs.	+ 357,000 lbs.
e	$0.48' = 5.8''$	$0.79' = 9.5''$
h/e	4.7	2.8
$\frac{R}{f_c}$	$\frac{187{,}000 \times 12}{53 \times 27^2 \times 600} = 0.097$	$\frac{281{,}000 \times 12}{53 \times 27^2 \times 780} = 0.112$
$\frac{d'}{h}$	$\frac{2.5}{27} = 0.09$	0.09
p per face (Fig. 37q)	....	0.27%

Use 7 1″ round bars in each face, making $p = \frac{7 \times 0.785}{1431} = 0.38\%$.

Section at Point 17.		
Dimensions, $h \times b$ 39.5 × 53	2094 sq. in.	2094 sq. in.
Moment	− 248,000 ft. lbs.	− 521,000 ft. lbs.
Thrust	+ 458,000 lbs.	+ 434,000 lbs.
e	$0.54' = 6.5''$	$1.2' = 14.4''$
$\frac{h}{e}$	6.1	2.8
$\frac{R}{f_c}$	$\frac{248{,}000 \times 12}{53 \times 39.5^2 \times 600} = 0.06$	$\frac{521{,}000 \times 12}{53 \times 39.5^2 \times 780} = 0.097$
$\frac{d'}{h}$	$\frac{2.5}{39.5} = 0.06$	0.06
p per face (Fig. 37q)	...	0.11%

This result is satisfactory.

Use 7 1″ round bars in each face, making $p = \frac{7 \times 0.785}{2094} = 0.26\%$.

Section at Point 19.		
Dimensions $h \times b$, 42.2 × 53	= 2237 sq. in.	2237 sq. in.
Moment	− 408,000 ft. lbs.	− 785,000 ft. lbs.
Thrust	+ 459,000 lbs.	+ 436,000 lbs.
e	$0.89' = 10.7''$	$1.8' = 21.6''$
$\frac{h}{e}$	4.0	2.0

$\frac{R}{f_c}$	$\frac{408{,}000 \times 12}{53 \times 42.2^2 \times 600} = 0.086$	$\frac{785{,}000 \times 12}{53 \times 42.2^2 \times 780} = 0.128$
$\frac{d'}{h}$	$\frac{2.5}{42.2} = 0.06$	0.06
p per face (Fig. 37q). .	. . .	0.39%

This result is satisfactory.

$$\text{Use 11 1'' round bars in each face, making } p = \frac{11 \times 0.785}{2237} = 0.39\%$$

A comparison of the stresses at the various sections indicates, as did the preliminary design, that the rib should have been made somewhat thinner in the haunches, thus effecting a saving in both concrete and steel amounting to several per cent. It will be noted that the section at Point 20 was not tested in the final design, although it was in the preliminary one.

The shear on the section at the crown is now figured, as explained under Item 26. We evidently have, allowing for the fact that the weight of the rib itself is distributed rather than concentrated,

Dead load, $22{,}800 - 25{,}400 \times \frac{1}{2} \times \frac{1}{2}$.	+ 16,500
Live load, $20{,}000\,(0.500 + 0.331 + 0.187 + 0.081 + 0.019)$. .	+ 22,400
Total.	+ 38,900

The net section at the crown is 24″ × 52″ = 1248 sq. in., so that the unit shear is

$$v = \frac{38{,}900}{1248} = 31 \text{ lbs. per sq. in.}$$

Since this shear is low, and as the arch rib follows the theoretic curve, it is evidently unnecessary to figure the unit shear at any other point. As an example of the method to be used for other points, however, the calculations for Point 12 will now be given.

The dead load shear on the left of this point is evidently,

$$S = 169{,}900 \times 0.908 - 308{,}200 \times 0.422 - 38{,}300 \times \tfrac{1}{2} \times \tfrac{1}{2} = + 14{,}500 \text{ lbs.};$$

while on the right it is,

$$S = 116{,}600 \times 0.908 - 308{,}200 \times 0.422 + 38{,}300 \times \tfrac{1}{2} \times \tfrac{1}{2} = - 14{,}700 \text{ lbs.}$$

The maximum positive live load shear on the left side of Point 12 will be obtained by loading all the load points to the right for which the ratio $\frac{V}{H_o}$ at Point 13 is greater than tan α at Point 12, or 0.464. Evidently we must load Points 12l, 8l, and 4l. We then have for the shear, by Equation 203,

$$S = 20{,}000\,[(0.919 + 0.813 + 0.669)\,0.908 - (0.434 + 0.831 + 1.175)\,0.422]$$
$$= 20{,}000\,(2.19 - 1.03) = +23{,}200 \text{ lbs.}$$

The maximum negative shear on the right side of Point 12 will be obtained by loading all the load points to the left—Points 16*l* and 12*l*—and those to the right for which the ratio $\frac{V}{H_o}$ at Point 11 is less than 0.464. We then have

$$S = 20{,}000\,[(-0.019 - 0.081 + 0.500 + 0.331 + 0.187 + 0.081 + 0.019)\,0.908 - (0.120 + 0.434 + 1.322 + 1.175 + 0.831 + 0.434 + 0.120)\,0.422]$$

$$= 20{,}000\,(0.924 - 1.880) = -19{,}300.$$

The shear from arch shortening is

$$S = -(-8{,}800 \times 0.422) = +3{,}700.$$

The shear for a 50° fall of temperature is

$$S = -(-29{,}700 \times 0.422) = +12{,}500,$$

and for a 30° rise of temperature,

$$S = -\frac{3}{5} \times 12{,}500 = -7{,}500.$$

We then have for the maximum shears at Point 12,

Loading	Positive to Left	Negative to Right
D	+ 14,500	− 14,700
L	+ 23,200	− 19,300
$A\ S$	+ 3,700	+ 3,700
T	+ 12,500	− 7,500
$D + L + A\ S$	+ 41,400	− 30,300
$D + L + A\ S + T$	+ 53,900	− 37,800

The area of the concrete at Point 12 is 33.86″ × 52 net = 1760 sq. in. The maximum unit shear, temperature not considered, is, therefore,

$$v = \frac{41{,}400}{1{,}760} = 24 \text{ lbs. per sq. in.};$$

while taking into account the effect of temperature, it becomes

$$v = \frac{53{,}900}{1{,}760} = 31 \text{ lbs. per sq. in.}$$

In a similar manner the unit shears at Points 8 and 19, when the effect of temperature is not considered, are found to be about 23 and 19 pounds per square inch, respectively; while, when the effect of temperature is taken into account, the corresponding figures are 33 and 24.

From the above figures it is evident that no shear reinforcement is required at any point.

THE CALCULATION OF STRESSES IN ARCH ABUTMENTS AND PIERS

Approximate Methods of Calculation for Abutments and Piers Carrying One Span Only

The approximate loads on the foundations of an arch abutment or pier carrying one span only can be computed by means of the formulæ previously given for the approximate calculation of stresses in arch ribs, with a few modifications. The following cases should be considered:

1. Maximum negative moment at springing.
2. Maximum positive moment at springing.
3. Full live load on arch rib.

For a shallow abutment, it will be sufficiently accurate to use the formulæ previously given for the moments and thrusts at the springing for Cases 1 and 2. For Case 3, it will be satisfactory to assume the moment at the springing as zero; and the corresponding thrust can be taken as that under dead load plus half-live load, plus $\frac{p\,l^2}{16\,r}\sec\beta$, p being the live load per lineal foot of rib.

For an abutment of considerable height, the thrusts calculated as just suggested are not accurate enough, either in direction or magnitude, although the values of the moments are sufficiently exact. The thrusts at the springing for such an abutment should be computed in the following manner:

The value of H_o under dead load, for an arch rib laid out for dead load only, is given by Equation 181; and the vertical component of the reaction at the springing is

$$V_s = H_o \tan\beta. \qquad \text{[Eq. 219]}$$

The value of H_o under dead load, for an arch rib laid out for dead load plus half-live load, is

$$H_o = \frac{p_o l^2}{48 r}(u + 5) - \frac{p\,l^2}{16 r}; \qquad \text{[Eq. 220]}$$

and the value of the vertical component of the reaction at the springing is

$$V_s = \frac{p_o l^2}{48\,r}(u + 5)\tan\beta - \frac{p\,l}{4}. \qquad \text{[Eq.221]}$$

The dead-load moment at the springing, for an arch laid out for dead load only, is zero; and for an arch laid out for dead load plus half-live load, it is negative and equal to one-half the difference between the positive and negative live-load moments at the springing, as found by Fig. 37*ee*—or say, equal to $-\,0.0023\,p\,l^2$.

For the maximum positive live-load moment at the springing, the live load will extend from the far end of the span to a point some distance past the centre, as can be seen from Table 37*h*. Assuming the load to extend over a length of 0.7 l from the far springing, that the moment at this latter point is zero (which will not be far wrong), and that the moment at the near springing is $+ 0.025\, p\, l^2$, we have as the value of the vertical component of the reaction at the near springing

$$V_s = \left(\frac{0.49 p l^2}{2} - 0.025\, p\, l^2\right) \div l = 0.22\, p\, l. \qquad \text{[Eq. 222]}$$

The value of H_o can be found by taking moments about the crown, M_o being taken equal to $+ 0.003\, p\, l^2$ (determined from Table 37h). We then have

$$H_o = (0.22\, p\, l \times \frac{l}{2} - 0.2\, p\, l \times 0.1\, l + 0.025\, p\, l^2 - 0.003\, p\, l^2) \div r$$
$$= 0.112\, \frac{p\, l^2}{r}. \qquad \text{[Eq. 223]}$$

For the maximum negative live-load moment at the springing, we may assume the live load to extend from the near support to a point about 0.35 l out. Assuming the moment at the near springing to be $- 0.02\, p l^2$, and that at the far one $+ 0.012\, p\, l^2$ (by Table 37h), we have, on taking moments about the far springing,

$$V_s = (0.35\, p\, l \times 0.825\, l + 0.02\, p\, l^2 + 0.012\, p\, l^2) \div l = 0.32\, p\, l. \qquad \text{[Eq. 224]}$$

The value of H_o can be found by taking moments about the near springing, assuming the moment at the crown to be $- 0.002\, p l^2$. We then find

$$H_o = \left\{\frac{.1225 p\, l^2}{2} - (0.35 - 0.32)\, p l \frac{l}{2} - 0.02\, p l^2 + 0.002\, p l^2\right\} \div r$$
$$= 0.028\, \frac{p l^2}{r}. \qquad \text{[Eq. 225]}$$

Since Case 3 will rarely be worse than one of the others, it will not be worth while to give more exact formulæ therefor. The approximate equations already presented are not seriously in error.

The formulæ previously given for the approximate calculation of stresses from arch shortening and temperature changes can be used in all cases. They give the value of H and the position of the point of contraflexure, from which the moments on the base or any other section can be computed. The stresses on the base should be figured without the effect of temperature changes, and also when they are included.

The calculation of stresses and moments on various sections can be done either analytically or graphically, the latter method being usually preferable.

Approximate Methods of Calculation for Piers Carrying Two Spans

For a pier between two arch ribs A and B, which may or may not be identical in form, we need to consider the following cases, in order to figure the maximum toe pressure on the side toward rib A:

1. Maximum negative moment at springing of rib A, and maximum positive moment at springing of rib B.
2. Dead load only on rib A, and full live load on rib B.
3. Full live load on both rib A and rib B.

For determining the maximum toe pressures on the side toward B, the conditions of loading on the two spans are to be reversed.

The formulæ necessary for the calculation of the above cases have all been stated previously. The formulæ for V_s and H_o given for use with high abutments should be employed for piers, unless they are unusually shallow. Case 1 assumes the use of two detached loadings, and rather high pressures can be allowed for it. The worst probable condition will be a little worse than Case 2, or than Case 1 with a load on one of the spans only. The effect of loads resting directly on the pier, and of the weight of the pier itself, must not be forgotten.

If it be desired to test horizontal sections through the pier shaft, the same three cases of loading should be considered as for figuring the foundation pressures.

A vertical section through the centre of the pier should also be tested, assuming maximum negative moment acting at the springings of both arches. The depth of section which will resist this moment is uncertain, and will vary somewhat in different cases; but it should usually be taken somewhat greater than the thickness of the arch rib at the springing.

In the case of the vertical section through the centre of the pier, the stresses should be figured both with and without considering the effects of temperature changes. If the two spans are alike, arch shortening and temperature stresses will have no effect upon the pressures on the foundation, or upon the stresses on any horizontal section of the shaft; but if the spans are different, these effects will have to be taken into account.

The graphical method of finding the moments and direct stresses will usually be found preferable to the analytic. In applying the graphical method for any section, it will be best to find first the resultant dead-load stress, and then combine therewith the live-load thrusts from the arch ribs. By following this procedure the probable effects of the various live loadings can be judged more easily, and hence the work can be completed with a minimum amount of labor.

It is occasionally desirable to determine to what extent an arch rib resting on a pier aids in taking up the thrust produced by loads on another rib resting on the same pier. In order to obtain approximate formulæ for this case, the pier will be considered as a vertical cantilever carrying a horizontal load at the arch springings; and it will be assumed that the springings of the ribs do not rotate as the top of the pier deflects. These assumptions are on the side of safety in all cases; and except for unusually flexible piers the error involved is small.

We first assume that the top of the pier has deflected sideways an arbitrary amount δ, the far ends of the two arch ribs not moving. The horizontal reaction H_p developed by the pier is

$$H_p = \frac{3\,E\,I\delta}{l_p^3} \qquad \text{[Eq. 226]}$$

in which I is the moment of inertia of the pier, and l_p its height. The thrust H_x developed in either of the arch ribs can be figured by multiplying the right hand member of Equation 193 by $\frac{l}{l}$, and then replacing $\omega\, t\, l$ (which expresses the change of length of the horizontal projection of the rib) by δ. We then obtain the formula,

$$H_x = C_t \frac{\delta\, E\, I_o}{r^2\, l}. \qquad \text{[Eq. 227]}$$

Now suppose that we have under consideration two ribs A and B, and that the rib A produces an unbalanced thrust (say from live load) of H on the pier between ribs A and B. If now we assign to the top of the pier an arbitrary deflection δ, and let H_A and H_B be the thrusts produced thereby in the two ribs, and H_p that on the pier, the total horizontal thrust developed is evidently $H_p + H_A + H_B$. The amount of thrust produced on the pier by the actual thrust H of rib A is evidently equal to $H \frac{H_p}{H_p + H_A + H_B}$; and the corresponding amount developed in rib B is $H \frac{H_B}{H_p + H_A + H_B}$. A tensile thrust of $H \frac{H_A}{H_p + H_A + H_B}$ is produced in rib A itself, which causes therein stresses similar to those from arch shortening or fall of temperature.

Should the pier and the adjoining rib at the far end of rib A be practically identical with those at the point under consideration, H_A in each of the above expressions is to be replaced by $2\,H_A$; for a similar movement δ will take place at the far end of the rib. Should, however, the pier and adjoining rib at the far end be different, so that the assignment of the movement δ at this point produces thrusts H'_p and H'_B in the pier and rib, the thrust on the rib B will be $H \dfrac{H_B}{H_p + H_B + H_A\left(1 + \dfrac{H_p + H_B}{H'_p + H'_B}\right)}$; that on the pier next to rib B will be $H \dfrac{H_p}{H_p + H_B + H_A\left(1 + \dfrac{H_p + H_B}{H'_p + H'_B}\right)}$; and that on the rib A will be $H \dfrac{H_A\left(1 + \dfrac{H_p + H_B}{H'_p + H'_B}\right)}{H_p + H_B + H_A\left((1 + \dfrac{H_p + H_B}{H'_p + H'_B}\right)}$.

When H'_p equals H_p and H'_B equals H_B, the coefficient of H_A becomes 2, which agrees with the statement previously made concerning the values obtained when the piers and adjoining ribs at the two ends are alike.

In most cases it will be found that H_p is so large as compared with H_A and H_B that only a small amount of thrust is supplied by the adjoining rib. The piers can sometimes be made somewhat light, so that this statement does not hold; but in this case the value of H_A will usually be large enough to set up rather high stresses in the rib in question. In general, therefore, the pier should be made stiff enough to take practically the entire thrust.

Exact Methods of Calculation

The same cases of loading as were given for the approximate design of arch abutments and piers can also be employed for the final design, using, of course, the exact values of the moments and thrusts at the springing. Except for shallow abutments, however, this method is hardly satisfactory; and the following procedure should generally be adopted.

The pier or abutment is first laid out to scale. The dead-load thrust at the springing (or springings) is then plotted, and combined with the weight of the portion of the pier above it. If there be any dead-load moment at the springing, the dead-load thrust will, of course, not pass exactly through the springing point. Next there are drawn reaction lines for unit loads at each load point of the arch. These can be laid out directly, since we know for each load the value of the horizontal thrust, the moment at the springing, and the vertical component of the thrust. By means of these lines it is possible to determine by inspection what load points are to be loaded in order to produce the maximum stresses at any given section of the pier. Such sections are then tested as may seem advisable. The live-load thrusts and moments on any section can be figured most easily by means of the reaction lines.

The sections to be tested have already been discussed in connection with the approximate calculations, and no further mention thereof is necessary here.

Fig. 37*kk* shows the reaction lines for the abutment of the arch the design of which has already been given in this chapter; and Fig. 37*ll* is a similar diagram for a pier supporting two equal arch ribs of one hundred and thirty-six (136) feet span and twenty-one (21) feet rise. The reaction lines in the latter figure are drawn for one rib only, since the two spans are alike. In the case of the abutment shown in Fig. 37*kk*, it was found necessary to compute the maximum pressures on both the front and the rear toes, and also the moment on a vertical section through the footing at about the centre. For the pier in Fig. 37*ll* the following sections were tested:

1. A vertical section through the centre of the pier, assuming the stresses to be resisted by a section five (5) feet thick. (The arch

ribs were four (4) feet thick at the springing.) This was figured for the maximum negative moment at the springings of the two arches, including the effects of arch shortening and fall of temperature.

2. Horizontal sections of the shaft at the bottom of the coping and at the bottom of the shaft. These sections were not affected by temperature changes or arch shortening, since the two arches were identical in form.

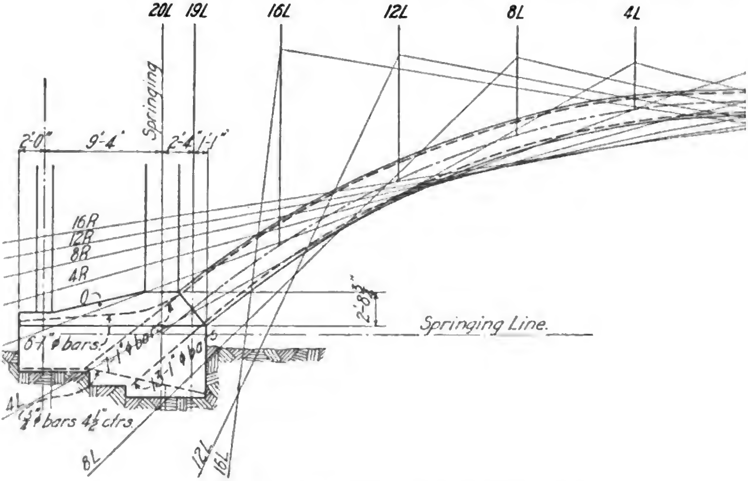


FIG. 37*kk*. Reaction Lines for an Arch Abutment.

3. The base of the pier. The pressures here were also unaffected by arch shortening and changes of temperature. The theoretic maximum pressure required the use of two detached loadings, one about twenty (20) feet long, the other about one hundred and fifteen (115) feet long. Since this condition was highly improbable, the effect of the load twenty feet long was ignored.

When it is desired to take into account the action of one rib resting on a pier in overcoming a portion of the thrust from another rib on the same pier, the approximate method previously outlined can frequently be used. Equation 227 is, however, to be replaced by the more exact expression,

$$H_x = \frac{E\,\delta \sec \beta \int \frac{d\,s}{I}}{2 \sec \beta \left[\int \frac{d\,s}{I} \int \frac{y^2\,d\,s}{I} - \left(\int \frac{y\,d\,s}{I} \right)^2 \right] + 2 \int \frac{\sec \alpha\, d\,s}{A} \int \frac{d\,s}{I}}. \quad \text{[Eq 228]}$$

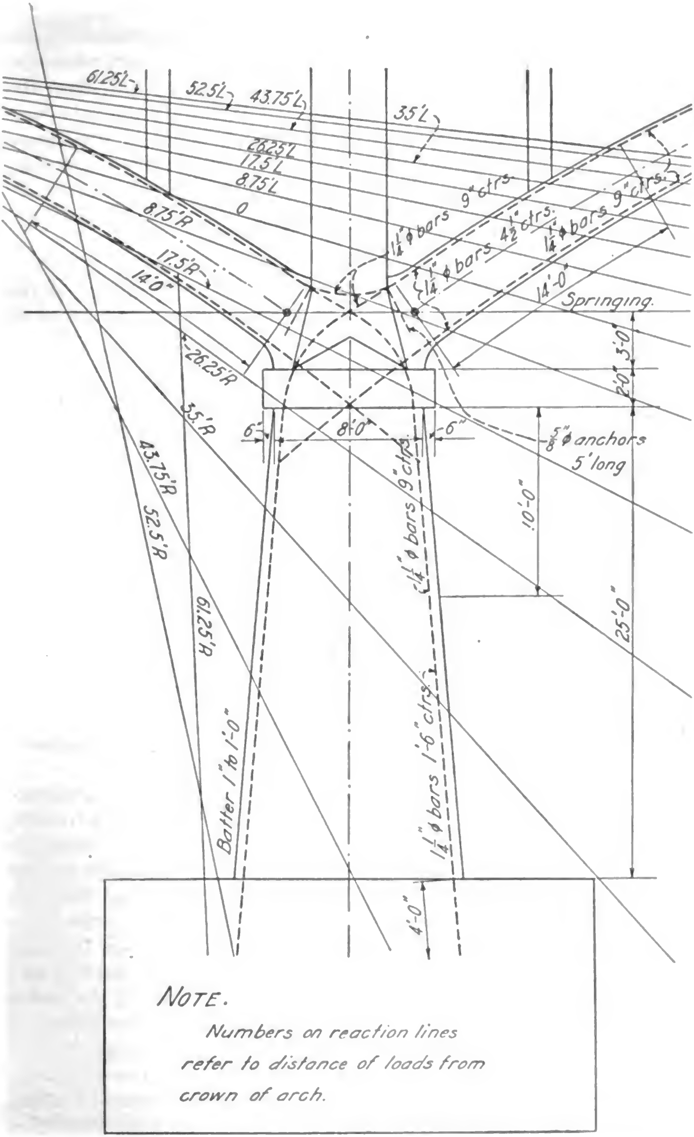


FIG. 37*ll*. Reaction Lines for a Pier Carrying Two Equal Arch Spans.

This latter formula is derived from Equation 212 by replacing $\omega t l$ by δ. The use of the approximate method is advisable even when the values of the thrusts found thereby are considerably in error, since the problem is statically indeterminate, and large variations from ideal conditions are likely to occur. Furthermore, a considerable error in the values of the thrusts will ordinarily produce very small errors in the foundation pressures As before stated, the approximate method errs on the side of safety. For comparatively flexible piers, it will be best to use more exact methods of calculation. In such a case, the stresses in the rib which is producing the unbalanced thrust should be carefully figured, as rather high values are likely to be caused by the deflections of the piers. Exact methods of analysis for elastic piers are to be found in Part III of Hool's "Reinforced Concrete Construction."

THE DESIGNING OF SLABS

The calculation of stresses in continuous slabs and in slabs reinforced in two directions has already been discussed in this chapter, as has also the question of the distribution of concentrated loads over slabs; so that no further treatment of the subject of the calculation of stresses is required. The design of a slab to suit any given bending moment can be made most easily by the use of Figs. 37*b* and 37*c*; and the unit shear and unit bond stresses can be found by means of Formulæ 125 and 126. The "Specifications for Design" on page 953 *et seq.* give the loads and the unit stresses which are to be employed; and they also cover certain features of both designing and detailing.

The depth of a slab designed for uniform loads only will be determined by the bending moment, as the unit shear will be low. When a slab carries concentrated loads, either the moment or the shear may govern, and both should be considered.

The type of slab most frequently used is continuous over the supports except at expansion points, where it is freely supported. In such slabs, the same amount of steel should be used at mid-span and over the supports. Several arrangements of the reinforcement are possible. One form which has been adopted largely in the author's practice is shown in Fig. 37*mm*. The arrangement of these bars in plan follows the order 1,2,3, 1,2,3, 1, etc., the bar 4 being placed over bar 1. The bars 1, 2, and 3 in the bottom really lie in the same plane, as do also the bars 2, 3, and 4 in the top; but it was necessary to separate them in making the sketch for the sake of clearness. The approximate positions of the points at which the bars 2 and 3 are to be bent are given; and these apply both to uniform and to concentrated loads. The laps in bars 1 can be short—say 12″; but those in bars 2 and 3 should be 40 diameters of the bar for deformed bars, and 50 diameters for plain bars. Bar 4 has sometimes been made continuous, like bar 1, to prevent temperature cracks. This is hardly worth while, except in very thick slabs having their top surfaces exposed, especially as it makes walking around on the steel during construction difficult. At an expansion joint bars 1 and 2 should be run to the end as shown. Bar 3 should be bent up as indicated if the shear is high, or stopped a short distance beyond the point noted for bending up if the shear is low. There should be transverse bars at the points indicated, to which every main bar should be wired or fastened by clips. In the centre portion of the slab these transverse bars should be placed from 12 inches to 18 inches apart in the bottom of the slab, unless, of course, more are required by the calculations. The transverse bars will usually be three-eighths ($\frac{3}{8}$) of an inch in diameter; but if the main bars are three-quarters ($\frac{3}{4}$) of an inch in diameter, the transverse bars should be one-half ($\frac{1}{2}$) inch.

The total weight of the steel required in the longitudinal reinforcement of the above type per lineal foot is about thirty-five (35) per cent more than the weight per lineal foot at the centre of each panel, allowing for laps and bends. This assumes each bar to be about three panels long. The weight of the transverse distribution steel per lineal foot of span, in slabs reinforced in one direction only, will be from fifteen (15) to thirty (30) per cent of the weight per lineal foot of the longitudinal reinforcement at the centre of the panel, the smaller value holding for slabs about one (1) foot thick, and the larger one for slabs only half as thick.

A very simple form of reinforcement is shown in Fig. 37*nn*. It should be used only when the shearing stresses are low; and it is, therefore, of limited application. The transverse steel should be arranged about as indicated, unless more be required by the calculations. The total weight per lineal foot of the main reinforcement for this type is about forty (40) per cent greater than the weight per lineal foot provided at the centre of the panel; and the weight of the transverse reinforcement per lineal foot of span, when used for distribution steel only, will be from fifteen (15) to thirty (30) per cent of the said quantity. In addition a considerable amount of steel in the form of chairs or spiders will be needed to support the top reinforcement.

The type of reinforcement shown in Fig. 37*oo* can also be used to advantage. It consists of one row of Bars "1," one row of Bars "2," one row of Bars "3," one row of Bars "4," one row of Bars "1," etc. It will be noted that all of these bars are alike—except at expansion points —Bars "1" and "3" being turned in one direction, and Bars "2" and "4" in the other. The total weight of the steel in the main reinforcement per lineal foot will be about thirty-five (35) per cent greater than the weight per lineal foot at the middle of the panel; while the amount of the transverse reinforcement, when used as distributing steel only, will be from fifteen (15) to thirty (30) per cent of the same quantity.

The details of slabs of various types are well illustrated in the before-mentioned article by Mr. Howard in the *Proc.* Am. Soc. C. E. for May, 1915.

In the case of slabs reinforced in two directions and carrying uniform loads, the rods in each direction should be spaced uniformly for the centre half of the slab; but near the edges the spacing may be increased to double that adopted in the centre portion. For such slabs which carry concentrated loading, the minimum spacing must be maintained for a width somewhat greater than that over which the load is assumed to distribute, and for at least two-thirds of the width of the slab. The spacing near the edges can be increased to double the minimum.

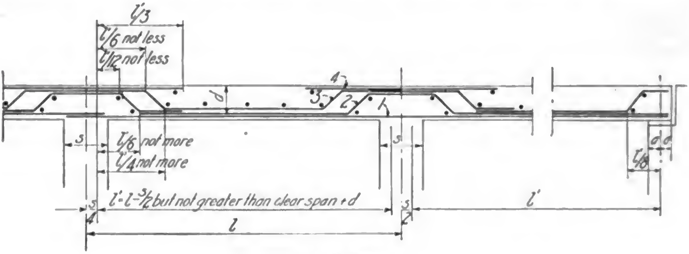


FIG. 37*mm*.

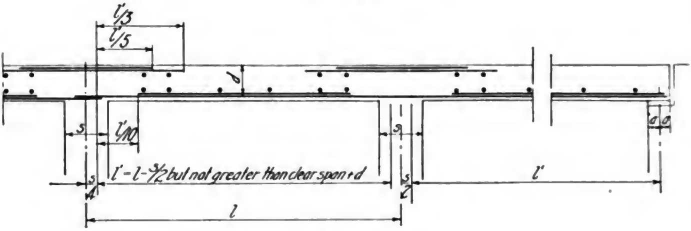


FIG. 37*nn*.

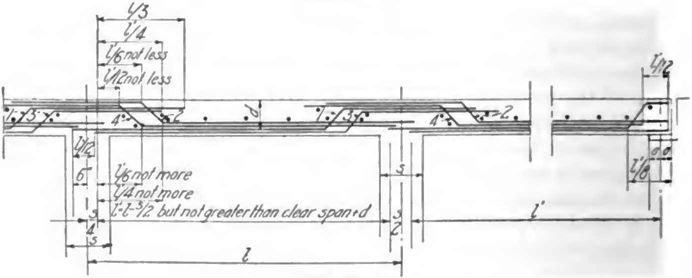


FIG. 37*oo*.

Arrangements of Slab Reinforcement.

In the case of slabs which are reinforced in one direction only, but are supported on all four sides, it will be proper to increase the spacing of the main reinforcement near the side supports. If the slab should be continuous or partly fixed at the said side supports, transverse bars may be needed in the top of the slab at these points to prevent cracking. The area of this transverse steel should be about one-third of that of the longitudinal steel; and it should extend from the support a distance of about one-third of the span length longitudinally of the structure.

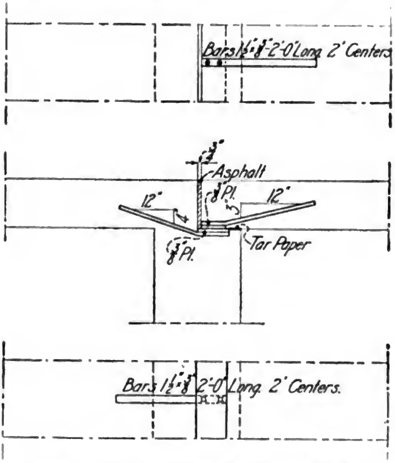

Bars 1½"×⅜"-2'-0" Long 2" Centers
¾"
Asphalt
⅜" Pl.
12"
12"
⅜" Pl.
Tar Paper
Bars 1½"×⅜" 2'-0" Long 2" Centers.

FIG. 37*pp*. Expansion Plates for Slabs.

At expansion joints it will be best to attach a steel plate to the under side of the slab and let it slide on a similar plate resting on the cross girder or other support. These plates should be efficiently anchored into the concrete. A detail which has been found very satisfactory for this purpose is shown in Fig. 37*pp*. In case the plates do not extend for the full width of the support, as in Fig. 37*pp*, tar paper should be used as there indicated.

The expansion joint should have an opening about 1 inch wide throughout, and should be filled with pure asphalt. Care should be taken in the detailing of expansion joints to see that all forms can be removed after the concrete has been poured. Tar paper, when used, is to be left in place.

The waterproofing of slabs has been discussed in Chapter XIX. Especial care is necessary at expansion joints, as the seepage of water through the floor is likely to discolor the concrete. Frequently, it will be advisable to provide small drain pipes to carry away any water that may collect in the expansion joints.

Construction joints in a slab should be made at an expansion joint or in the centre of a panel. Where the slab is of sufficient thickness, a tongue-and-groove joint should be provided.

A slab should always rest on top of a girder whenever possible. If it should be found necessary to place the supporting girder above the slab, vertical stirrups, well bonded into both the slab and the girder, must be used to carry the entire load of the said slab.

THE DESIGNING OF CANTILEVER BEAMS AND CROSS-GIRDERS

Cantilever beams in reinforced concrete construction are nearly always of varying depth, being much deeper at the supports than at the outer ends. The critical section for bending moment will ordinarily be at the edge of the support; but in large beams it will be necessary to figure the moments at other sections as well, in order to determine the points for stopping some of the bars. If the load on the cantilever is uniformly distributed along its length, the maximum unit shear will occur at the support, and the maximum diagonal tension is to be figured on a section located at a distance equal to half the depth of the cantilever at the support from the edge of the latter; but if there is a concentration at the outer end—as from a fascia girder and handrail—the critical section for shear may be just inside of the said concentration.

The design of a cantilever for bending moment can be best accomplished by means of Fig. 37*j*. The unit shear and unit bond stress can be figured by means of Equation 139 and the accompanying instructions. Shear reinforcement, if required, can be designed by means of Fig. 37*r*. The unit stresses to be employed can be taken from the "Specifications for Design" given on page 953 *et seq.*, as can also the minimum spacings and edge distances of the bars.

As a cantilever is a conspicuous member of a bridge, its dimensions are frequently determined by the requirements of æsthetics, rather than those of strength. When employed to support the outer portions of a roadway—their ordinary use—those over the piers may be quite heavy. The intermediate ones will generally be twelve (12) inches or fifteen (15) inches thick. The bottom line is usually made a flat curve, although sometimes it is laid out straight, while in other cases it is curved sharply. The latter detail gives the best appearance; but it requires the cantilever to be rather deep, and in many structures it cannot well be used for this reason.

The reinforcement is simple, ordinarily consisting of straight bars in the top which pass over the support and extend into the construction beyond. Some of the bars should run to the outer end of the beam; but others are usually stopped off, if the unit shear is low, or are bent down to serve as shear reinforcement if any be needed. Vertical stirrups, when required, should pass around the top steel and have hooks at their lower ends. The use of steel in the compression side is unnecessary. The top steel is frequently placed near the top of the slab which the cantilever supports; but this should not be done unless the slab and cantilever are to be poured at one operation.

The most usual type of cross-girder is a beam of nearly constant depth, supported at its ends on girders or columns. It is nearly always monolithic with the said girders or columns, so that its ends are partly restrained; and very frequently it is continuous with a cantilever beam at each end. For the calculation of stresses in the cross-girder from loads on the cross-girder and cantilevers, it will generally be best to consider it as freely supported on the columns or girders. If it be desired to figure the wind stresses in a bent composed of a cross-girder and two columns, the cross-girder and columns should be considered continuous. The moments in the cross-girder should be figured for three cases of loading, viz.:

1. Dead load on cross-girder and cantilevers.
2. Live load on cross-girder.
3. Live load on both cantilevers.

Moment curves for the maximum positive and maximum negative moments should then be drawn. The maximum shear should be figured on a section located at a distance equal to half the depth of the cross-girder from the edge of the support, with live load on the cross-girder and the adjacent cantilever.

The cross-girder can be designed as a T-beam if it and the slab are to be poured at one operation, otherwise as a rectangular beam. Figs. 37*h* and 37*h′* can be used in the first case, and Fig. 37*b* in the second. The unit shearing and unit bond stresses can be computed by means of Equations 125 and 126 in either case, replacing b by b' if the cross-girder be considered a T-beam. Shear reinforcement, if required, can be designed by means of Fig. 37*r*. It will usually be needed. The unit stresses to be employed are given in the "Specifications for Design" in this chapter, as are also the minimum spacings and edge distances of the reinforcing bars.

The bottom reinforcement for a cross-girder of the above type will consist of one or two layers of bars. Part of them will run the full length, while others will be bent up to serve as shear reinforcement. The top reinforcement will usually be made by extending the reinforcement from the cantilevers over into the cross-girder. Part of it should be carried throughout the top of the cross-girder, and the rest of it bent down to serve as shear reinforcement. It will generally be found possible to use some of these latter bars for the bottom reinforcement. If stirrups are needed for shear reinforcement, they should pass around the top steel, and be hooked around the lower steel; while if they are used merely to support the lower steel, they should pass around it, and bend outward at the top so that the ends rest on the forms for the slabs.

Another type of cross-girder frequently used is similar to the above, but has no cantilevers at the ends. It should be designed as a simply supported beam, unless it is carried on excessively heavy columns which will make it nearly fixed at the ends. The designing and detailing will be similar to that for the cross-girder with cantilevers, except that the top reinforcement will not be required, unless the ends are nearly fixed by the columns; and the stirrups must in all cases pass around the bottom steel.

If there are three or more lines of main girders or columns, the cross-girders should be made continuous over the intermediate ones and the stresses figured accordingly, taking due account of the moments from the cantilevers, if there be any. It will usually be necessary to make exact calculations for the moments, as the use of standard coefficients may give results too greatly in error. Curves for maximum positive and negative moments should be drawn, and steel should be provided as required. The designing of steel in continuous girders is discussed in the next section of this chapter.

The subject of stresses in cross-girders caused by the stiffness of columns or by wind will be discussed in connection with the columns, as it is usually these rather than the cross-girders that are affected by such stresses. In most cases no special reinforcement will be required in the cross-girders.

The details of several different forms of cantilevers and cross-girders are well illustrated in Mr. Howard's paper on the Twelfth Street Trafficway Viaduct at Kansas City. On one span of this structure it was necessary to keep the total thickness of the floor of the lower deck down to two (2) feet, although the longitudinal girders were spaced thirty-six (36) feet on centres. The problem was solved by the use of shallow steel cross-girders (not illustrated in Mr. Howard's paper) encased in concrete. These were spaced six (6) feet apart, and carried a special thin slab. Special structural steel brackets were used at the ends to deliver their loads to the main girders, which were through-girders.

THE DESIGNING OF MAIN GIRDERS

Main girders in reinforced-concrete bridges are usually continuous over the supports except at expansion joints, where they are simply supported, these joints in most cases being located at every third or fourth support. The methods of calculating moments and shears in such girders have already been treated, and no further explanations are necessary here.

The design of any section of a girder to suit a given bending moment can be accomplished by means of formulæ and diagrams already given. The section at a support must be proportioned as a rectangular beam, for which purpose Fig. 37*b* can be employed. The section at the centre of a span can be designed in the same manner; but if the girder carries a slab which is to be poured simultaneously with it, T-beam action can be assumed, and Fig. 37*h* or Fig. 37*h′* employed. The bottom flanges of girders are frequently arched for æsthetic reasons; and when the amount of curvature is large, it will be best to use Fig. 37*j* for rectangular beams of varying depth in designing the sections at the supports.

Shearing stresses in main girders are important, and frequently determine the dimensions thereof. As previously explained, the vertical section at the edge of a support on which a girder is simply supported is to be figured for diagonal tension; but a similar section at the edge of a support over which a girder is continuous is to be designed for pure shear only, the critical section for diagonal tension being considered to be located at a distance equal to half the depth of the girder from the edge of the support. If the bottom surface of the girder is nearly horizontal, Equations 125 and 126 are to be employed, substituting b' for b if the girder is considered to be a T-beam. If the bottom surface is inclined considerably, Equation 139 should be used.

In figuring the unit bond stresses, Equation 126 evidently applies directly only to girders in which all rods are straight and extend over the full length of the girder. In the case of girders having some of the bars bent up or stopped off, the allowable bond stress will theoretically never be exceeded if all bars extend past any section at which they are needed far enough to be fully developed at the said section. However, it has been found by tests that if bars are stopped at a point where the steel stresses are high, excessive bond stresses and a tendency to slippage occur; so that ordinarily they should be bent up into the shear region rather than stopped off.

The "Specifications for Design" on page 953 *et seq.* give the unit stresses to be employed, and also cover such points as the permissible spacing of bars, edge-distances, etc.

The arrangement of the reinforcement in continuous girders requires considerable care. In order to do this to the best advantage, curves of maximum positive and negative moments in each span should first be drawn. The amount of steel required at the centre of each span and over each support should then be computed, and from this the size, number, and arrangement of the bars at each of these sections should be determined. The side elevation of the girder should then be laid out to scale, and the points for the bending up of the various bars determined by means of the moment diagram. Bars bent up from the bottom reinforcement should be used in the reinforcement over the supports as far as possible; and they should be arranged so as to reinforce for diagonal tension in the most effective manner. Bars should be extended some distance past the points where they could theoretically stop, in order to ensure that the bond stresses will be low. This procedure will also keep the unit stresses in the steel low, which will strengthen the girder considerably in diagonal tension.

When stirrups are required as web reinforcement, those in the central portion of the girder should be of the type shown in Fig. 37*tt*; while those in the end portions, where the moment is negative, should be similar but inverted. When stirrups are not needed as web reinforcement, it will still be best to use them spaced about three (3) or four (4) feet centres, in order to support the bottom steel. They should be of the type shown in Fig. 37*tt*. Stirrup bars in light girders should be ⅜ inch in diameter, and in heavy girders ½ inch.

To illustrate the method of designing the reinforcement of a continuous girder, the calculations for the three-span girder shown in Fig. 37*qq* will now be given in detail.

The girders in the three spans will be assumed to be of the same thickness and depth, so that the moment of inertia will be constant throughout.

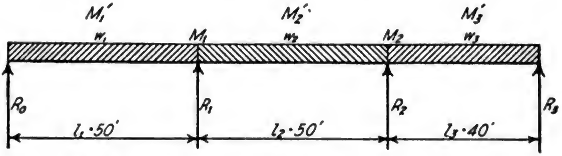


FIG. 37*qq*. Sketch of a Three-Span Continuous Girder.

The panel lengths of the floor-system will be assumed to be so short that the entire load can be considered uniformly distributed without introducing any error of importance.

The moment coefficients for M_1 and M_2 are determined by means of Fig. 37*v*. Entering first with $c_1 = \frac{50}{50} = 1$, and $c_3 = \frac{40}{50} = 0.8$, we find

$$M_1 = -0.067\, w_1 l_1^2 - 0.048\, w_2 l_2^2 + 0.015\, w_3 l_3^2.$$

Again, letting $c_1 = 0.8$, and $c_3 = 1.0$, we have

$$M_2 = 0.019\, w_1 l_1^2 - 0.056\, w_2 l_2^2 - 0.060\, w_3 l_3^2.$$

Substituting the values of l_1, l_2, and l_3 in these expressions, we find

$$M_1 = -167\, w_1 - 120\, w_2 + 24\, w_3,$$

and $M_2 = 47\, w_1 - 140\, w_2 - 96\, w_3.$

We next figure the moment at the centre of each span, assuming it to be simply supported, and obtain the values,

$$M'_1 = \tfrac{1}{8} \times 50^2 \times w_1 = 313 w_1,$$

$$M'_2 = 313\, w_2,$$

and $M'_3 = \tfrac{1}{8} \times 40^2 \times w_3 = 200\, w_3.$

The dead load per lineal foot of girder will be taken as 15,000 pounds, and the live load 7,000 pounds. We have, therefore, the following moments:

Dead Load,

$M_1 = 15{,}000\ (-167 - 120 + 24) = -3{,}950{,}000$ ft. lbs.
$M_2 = 15{,}000\ (47 - 140 - 95) = -2{,}840{,}000$ " "
$M'_1 = 15{,}000 \times 313 = +4{,}700{,}000$ " "
$M'_2 = 15{,}000 \times 313 = +4{,}700{,}000$ " "
$M'_3 = 15{,}000 \times 200 = +3{,}000{,}000$ " "

Live load on span l_1,

$M_1 = -167 \times 7000 = -1{,}170{,}000$ ft. lbs.
$M_2 = 47 \times 7000 = +330{,}000$ " "
$M'_1 = 313 \times 7000 = +2{,}190{,}000$ " "

Live load on span l_2,

$M_1 = -120 \times 7000 = -840{,}000$ ft. lbs.
$M_2 = -140 \times 7000 = -980{,}000$ " "
$M'_3 = 313 \times 7000 = +2{,}190{,}000$ " "

Live load on span l_3,

$M_1 = 24 \times 7000 = +170{,}000$ ft. lbs.
$M_2 = -96 \times 7000 = -670{,}000$ " "
$M'_3 = 200 \times 7000 = +1{,}400{,}000$ " "

We then proceed to plot the moment diagrams. For span l_1, the maximum positive moment occurs with live load on spans l_1 and l_3. Under these conditions, we have

$M_1 = -3{,}950{,}000 - 1{,}170{,}000 + 170{,}000 = -4{,}950{,}000$ ft. lbs.
and $M'_1 = 4{,}700{,}000 + 2{,}190{,}000 = +6{,}890{,}000$ " "

The three spans are then laid off to some convenient scale, as in Fig. 37*rr*. We next lay off a line vertically upward at R_1, representing the moment 4,950,000 foot pounds to any desired scale. The moment in the span l_1, due to the moment M_1 acting at R_1, varies uniformly from $-4{,}950{,}000$ at R_1 to zero at R_o, and is, therefore, represented by a straight line. The combined effect of the moment M_1 and the load on the span l_1 can be found by plotting the parabola of moments from the said straight line as a reference line rather than from the horizontal line, making the mid-ordinate 6,890,000 foot pounds. The ordinates between the resulting curve and the horizontal line will then represent the moments in the span.

We must next draw the curve for maximum negative moments near R_1. For this purpose we put the live load on spans l_1 and l_2, whence we find

$M_1 = -3{,}950{,}000 - 1{,}170{,}000 - 840{,}000 = -5{,}960{,}000$ ft. lbs.
and $M'_1 = +6{,}890{,}000$ " "

We next lay off $M_1 = 5{,}960{,}000$ foot pounds vertically upward at R_1 to the same scale as in the first case, and draw the moment diagram in the same manner.

We then proceed to draw the curve for maximum negative moments in the central part of span l_1. For this case we put live load on span l_2 only, so that we have

$M_1 = -3{,}950{,}000 - 840{,}000 = -4{,}790{,}000$ ft. lbs.
and $M'_1 = +4{,}700{,}000$ " "

We must next draw the moment diagrams for span l_2. For maximum positive moments, we put live load on span l_2 only, whence we have

$M_1 =$ $-4{,}790{,}000$ ft. lbs.
$M_2 = -2{,}840{,}000 - 980{,}000 = -3{,}820{,}000$ " "
and $M'_2 = 4{,}700{,}000 + 2{,}190{,}000 = +6{,}890{,}000$ " "

For maximum negative moments near the centre of span l_2, we put live load on spans l_1 and l_3, giving the moments

$M_1 =$ $-4{,}950{,}000$ ft. lbs.
$M_2 = -2{,}840{,}000 - 670{,}000 + 330{,}000 = -3{,}180{,}000$ " "
and $M'_2 =$ $+4{,}700{,}000$ " "

For maximum negative moments near R_1, we load spans l_1 and l_2, whence we have

$M_1 =$ $-5{,}960{,}000$ ft. lbs.
$M_2 = -2{,}840{,}000 - 980{,}000 + 330{,}000 = -3{,}490{,}000$ " "
and $M'_2 =$ $+6{,}890{,}000$ " "

For maximum negative moments near R_2, we must place live load on spans l_2 and l_3; and the resulting moments are

$M_1 = -3{,}950{,}000 - 840{,}000 + 170{,}000 = -4{,}620{,}000$ ft. lbs.
$M_2 = -2{,}840{,}000 - 980{,}000 - 670{,}000 = -4{,}490{,}000$ " "
and $M'_2 =$ $+6{,}890{,}000$ " "

The moment curves for each of the four conditions of loading are then drawn.

It is next necessary to check to see that the positive moment at centre of span l_2 is taken large enough. The minimum allowable value is $\frac{22{,}000 \times 50^2}{20} = 2{,}750{,}000$ ft. lbs.; and as the plotted moment is only 2,600,000 ft. lbs., the steel area must be figured for the larger value.

The moment diagrams for span l_3 are now to be prepared. For maximum positive moments, we load spans l_1 and l_3, whence we have

$M_2 =$ $-3{,}180{,}000$ ft. lbs.
and $M'_3 = 3{,}000{,}000 + 1{,}400{,}000 = +4{,}400{,}000$ " "

For maximum negative moments near R_2, we load spans l_2 and l_3, whence we find

$M_2 =$ $-4{,}490{,}000$ ft. lbs.
and $M'_3 =$ $+4{,}440{,}000$ " "

For maximum negative moments near the centre of l_3, we load span l_2 only, and the resulting moments are

$M_2 =$ $-3{,}820{,}000$ ft. lbs.
and $M'_3 =$ $+3{,}000{,}000$ " "

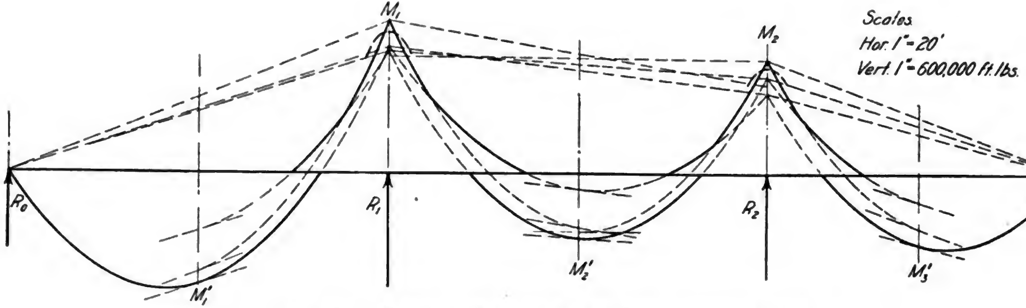


Fig. 37*rr*. Moment Diagram for a Three-Span Continuous Girder.

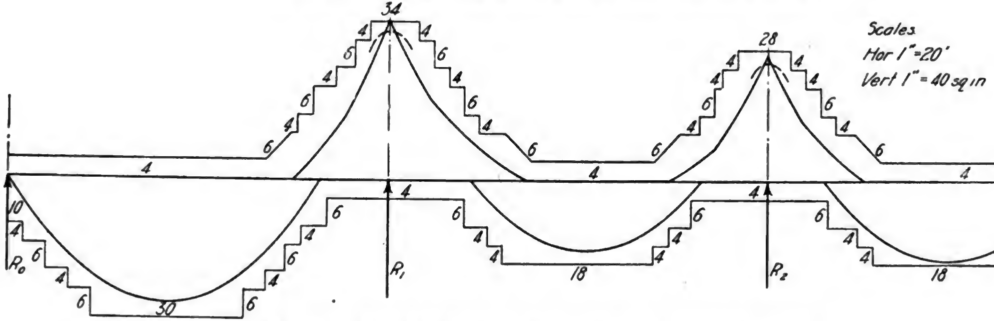


Fig. 37*ss*. Steel-Area Diagram for a Three-Span Continuous Girder.

The moment diagrams for each of the three conditions of loading are then plotted.

As an aid in drawing the parabolas, it should be remembered that the slope of the curve at mid span is parallel to the reference line from which it is plotted. This is illustrated in Fig. 37*rr*. Only the parts of the parabolas shown by full lines need be drawn in, as the dotted portions do not determine steel areas.

On account of the width of the supports R_1 and R_2, the moment diagram at those points is not really of the shape just determined. The dotted lines show the approximate form it will actually have.

The maximum shear must also be figured. It will evidently occur in the span l_1, at the edge of the support R_1. Let us assume the distance from R_1 to the edge of the support to be three (3) feet. The maximum shear at this point will occur when spans l_1 and l_2 are loaded, and its value is

$$V = 22{,}000\ (25 - 3) + \frac{5{,}960{,}000}{50} = 603{,}000 \text{ lbs.}$$

The section of the girder will be determined by the moment at R_1 or at the centre of the span l_1, or else by the maximum shear. Assuming the girder to be four (4) feet wide, and that there is no T-beam action, we find the following required depths:

$$\text{For moment } M_1,\quad d = \sqrt{\frac{5{,}960{,}000}{4 \times 120}} = 111'',$$

$$\text{for moment in } l_1,\quad d = \sqrt{\frac{4{,}620{,}000}{4 \times 95}} = 110'',$$

$$\text{and for maximum shear,}\quad d = \frac{603{,}000}{120 \times 48 \times \frac{7}{8}} = 119''.$$

Assuming $d = 120''$ or $10'$, and $j = \frac{7}{8}$, the steel area required for the moment M_1 is

$$A = \frac{5{,}960{,}000}{10 \times \frac{7}{8} \times 16{,}000} = 42.6 \text{ sq. in.,}$$

which calls for 35 1¼-inch round bars having a total area of 43.0 square inches. Evidently three rows of bars at ten (10) each will be sufficient except just at R_1, so that the gross depth of the girder below the slab must be about $119'' + 4'' + 2\frac{1}{2}'' = 125\frac{1}{2}''$. We shall, therefore, make the gross depth 126″, or 10′ 6″, and call the effective depth 120″, or 10′. The quantity j will be $\frac{7}{8}$ or more at all points, and this value will be used in computing the steel area. The required area at any point will, therefore, be

$$A = \frac{M}{16{,}000 \times \frac{7}{8} \times 10} = \frac{M}{140{,}000}$$

The required areas of the top and bottom reinforcement are next obtained by the formula just given. The smooth curves in Fig. 37*ss* give the resulting values.

The reinforcement is now to be designed. Deformed bars 1¼ inches in diameter will be employed, putting ten bars in each row, as has been previously stated. There will be needed 27 bars at the centre of span l_1, 16 at the centre of span l_2, 17 at the centre of span l_3, 34 at R_1, and 27 at R_2. The numbers used will be 30, 18, 18, 34, and 28, respectively. The points at which bars are to be bent up or stopped off must next be determined. All bent-up bars will be inclined at about 45°, and the horizontal projection of the bent portion will be taken as nine (9) feet. Fig. 37*tt* shows the arrangement adopted; and the stepped lines in Fig. 37*ss* indicate the areas provided at various points. The vertical portions of these lines show the points at which the bars bend up, and the sloped portions indicate the gradual development of bars which stop without being bent up. The bars are assumed not to resist any of the moment after being bent, which assumption is, of course, on the side of safety.

Since the unit shear exceeds forty (40) pounds per square inch at some points, web reinforcement is required. The critical sections at R_0 and R_3 are at the edges of the supports, or say one (1) foot from the centres thereof. The critical sections at R_1 and R_2 are located $\frac{d}{2}$ from the edges of the supports, or eight (8) feet from the centres. The amount of shear that can be carried by the concrete and bent-up bars will be determined, and stirrups will be supplied if necessary.

The end of span l_1 next to R_1 will first be considered. The shears at various sections, due to full live load on spans l_1 and l_2, will first be figured, and then the effect of removing the portion of the live load between the section and R_1 will be computed on the assumption that the girder is simply supported at R_1. We then have:

At edge of support (see previous figures).

$$V = 603{,}000 \text{ lbs.}$$

$$v = 603{,}000 \div 48 \times 120 \times 7/8 = 603{,}000 \div 5{,}040 = 120 \text{ lbs. per sq. in.}$$

At 8′ from R_1.

$$V = 603{,}000 - 5 \times 22{,}000 + \frac{8^2 \times 7{,}000}{2 \times 50} \qquad = 498{,}000 \text{ lbs.}$$

$$v = 498{,}000 \div 5{,}040 \qquad = 99 \text{ lbs. per sq. in.}$$

At 13′ from R_1.

$$V = 603{,}000 - 10 \times 22{,}000 + \frac{13^2 \times 7{,}000}{100} \qquad = 395{,}000 \text{ lbs.}$$

$$v = 395{,}000 \div 5{,}040 \qquad = 79 \text{ lbs. per sq. in.}$$

At 20′ from R_1.

$$V = 603{,}000 - 17 \times 22{,}000 + \frac{20^2 \times 7{,}000}{100} \qquad = 257{,}000 \text{ lbs.}$$

$$v = 257{,}000 \div 5{,}040 \qquad = 51 \text{ lbs. per sq. in.}$$

At 25′ from R_1.

$$V = 603{,}000 - 22 \times 22{,}000 + \frac{25^2 \times 7{,}000}{100} \qquad = 163{,}000 \text{ lbs.}$$

$$v = 163{,}000 \div 5{,}040 \qquad = 32 \text{ lbs. per sq. in.}$$

The shear diagram for this portion of the beam is now drawn to scale, as shown at the top in Fig. 37*tt*. The unit shear carried by the concrete, 40 pounds per square inch to a point 8 feet from R_1, and thence increasing uniformly to 120 at the edge of the support, is then laid off. Evidently shear reinforcement is needed from the edge of the support to a point 23 feet from R_1. The unit shear cared for by the various bent-up bars is next to be computed as follows, using Fig. 37*r*:

Bars bent down 2′ 6″ from R_1.

$$m_i = \frac{4}{4} = 1 \text{ per ft. width.}$$

$$s_i = \frac{4}{10}(42'' + 24'') = 26.4''$$

$$\therefore v_i = 66 \text{ lbs. per sq. in.}$$

Bars bent down 4′ 6″ from R_1.

$$m_i = \frac{6}{4} = 1.5$$

$$s_i = \frac{6}{10}(24'' + 30'') = 32.4''$$

$$\therefore v_i = 80 \text{ lbs. per sq. in.}$$

Bars bent down 7′ 0″ from R_1.

$$m_i = 1$$

$$s_i = \frac{4}{10}(30'' + 36'') = 26.4''$$

$$\therefore v_i = 66 \text{ lbs. per sq. in.}$$

Bars bent down 10′ 0″ from R_1.

$$m_i = 1.5$$

$$s_i = \frac{6}{10}(36'' + 24'') = 36''$$

$$\therefore v_i = 72 \text{ lbs. per sq. in.}$$

Bars bent down 12′ 0″ from R_1.

$$m_i = 1$$

$$s_i = \frac{4}{10} \times 24'' \times 2 = 19.2''$$

$$\therefore v_i = 90 \text{ lbs. per sq. in.}$$

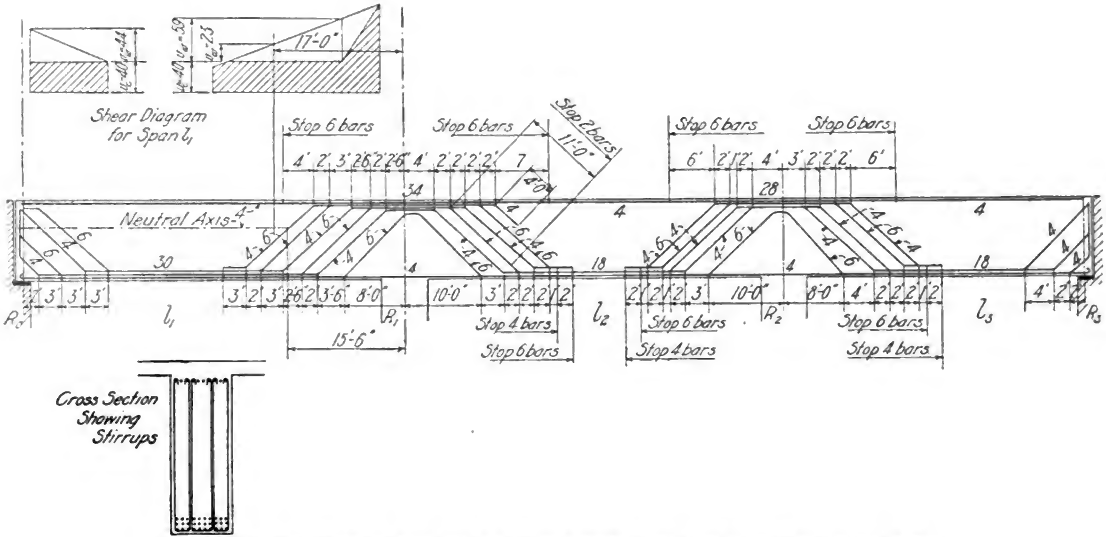


FIG. 37*u*. Shear Diagram and Details of Reinforcement for a Three-Span Continuous Girder.

Since the maximum value of v_w is 59 pounds per square inch, it is evident that the bent-up bars will care fully for the shear in the portion of the beam covered by them. The last row of these bars is located beyond the point of contraflexure, so that we need to test the shearing strength here only between the bottom reinforcement and the neutral axis. This last row cuts the neutral axis about fifteen and one-half (15.5) feet from R_1, and since it provides a shearing strength of 90 pounds per square inch when assumed to reinforce for a space 19.2 inches wide, it will evidently be safe to consider it to reinforce for a width somewhat greater than this, so that vertical stirrups will be needed first at a point about seventeen (17) feet from R_1. The value of v_w at this point is 23 pounds per square inch. Stirrups will be made of ½″ round bars, arranged as shown in Fig. 37*tt*. There will be six (6) vertical bars per stirrup, so that the value of m_v per foot of width will be 1.5. From Fig. 37*r*, we find the required spacing to be 12″. To be on the safe side, this spacing will be used from R_1 to the centre of the span. Near the centre of the beam, the bars will be placed as shown in Fig. 37*tt*; but those within ten feet of R_1 will be inverted. A length of embedment of 30 × ½″. or 15″, is sufficient for the development of the stirrup bars; and since the compression area is nearly four feet deep, the hooks on the ends are essential only near the point of contraflexure, where either the top or the bottom face may be the tensile one.

If the last row of bars had not been located beyond the point of contraflexure, the critical section for shear would have been at the top of the beam. This last row of bars cuts a horizontal section here only 12 feet from R_1, so that stirrups would first have been needed 13.5 feet from R_1. The value of v_w at this point is 37 pounds per square inch, so that the required stirrup spacing here would have been 8″.

The end of span l_1 next to R_o will now be considered. Covering spans l_1 and l_3 with live load, and proceeding as before, we find for the values of shear at various sections:

At edge of support—1′ from R_o.

$$V = 22{,}000(25 - 1) - \frac{4{,}950{,}000}{50} = 429{,}000 \text{ lbs.}$$

$$v = 429{,}000 \div 5040 = 84 \text{ lbs. per sq. in.}$$

5′ from R_o.

$$V = 429{,}000 - 22{,}000 \times 4 + \frac{7000 \times 5^2}{2 \times 50} = 343{,}000 \text{ lbs.}$$

$$v = 343{,}000 \div 5040 = 68 \text{ lbs. per sq. in.}$$

10′ from R_o.

$$V = 429{,}000 - 22{,}000 \times 9 + \frac{7000 \times 10^2}{2 \times 50} = 238{,}000 \text{ lbs.}$$

$$v = 238{,}000 \div 5040 = 47 \text{ lbs. per sq. in.}$$

The shear diagram is then laid off, as shown in Fig. 37*tt*, and the unit shear carried by the concrete, 40 pounds per square inch, is plotted thereon. The unit shears cared for by the various rows of bent-up bars are then figured as follows:

Bars bent up 2′ from R_o.

$$m_i = 1.$$

$$si = \frac{4}{10} \times 36'' \times 2 = 28.8.''$$

$$v_i = 60 \text{ lbs. per sq. in.}$$

Bars bent up 5′ from R_o.

$$m_i = 1.5.$$

$$s_i = \frac{6}{10} (36'' + 36'') = 43.2''.$$

$$\therefore v_i = 60 \text{ lbs. per sq. in.}$$

Evidently the same result is true for the other two rows. The bent-up bars are seen to provide sufficient web reinforcement except in the portion from seven (7) to eleven (11) feet from R_o, where a small amount of vertical steel will be required. To be on the safe side, we shall use the same stirrups as were employed for the other half of the beam, spaced 2′−0″ apart, from R_o to the middle of the span.

The web reinforcement for the other spans is now to be designed in the same manner as for span l_1.

Hooks on the bent-up bars are unnecessary. At the ends of the girder, the bars are carried up to the top in order to secure a proper length of embedment; while those which end on the tensile side of the beam are turned horizontally and extended 3′−0″ or practically 30 diameters.

The unit bond stresses will be highest near the expansion ends. At the section one foot from the centre of R_o, there are only ten (10) bars in the bottom reinforcement, although there are four (4) inclined bars about a foot above the bottom. Neglecting the effect of these inclined bars, we find for the unit bond stress

$$u = \frac{429{,}000}{120 \times \frac{7}{8} \times 10 \times 3.14 \times 1.25} = 105 \text{ lbs. per sq. in.}$$

Since the four inclined bars are about as effective for resisting moment as two horizontal bars in the bottom, the correct value of u is evidently not far from $105 \times \frac{10}{12} = 88$ lbs. per sq. in., which is satisfactory. In order to strengthen this section as much as possible, however, the ten (10) straight bars will be hooked at the ends.

The bars in the top which are continuous from one span to the next will have to be spliced near the centres of the spans, and the straight bars in the bottom, over the supports. The bent-up bars will be spliced in the diagonal portions. By this arrangement no splices occur at point of maximum stress or at points where there are a great number of bars.

The shoes at the expansion ends of a girder of this type should be thoroughly anchored into the concrete of the girder, in order to prevent them from tearing loose when the girder contracts; and the plates on which the shoes slide must be anchored equally well into the concrete of the piers or abutments. The anchors should be designed for a pull equal to twenty-five (25) per cent of the load on the shoe. Fig. 37*uu* shows the type of detail used on the Twelfth Street Trafficway for this purpose.

Construction joints should, preferably, be at the expansion points; but when this is impossible, they should be located near the centres of the spans. In the latter case a substantial vertical bulkhead should be constructed, and some provision for carrying the shear at the joint should be provided. This can be accomplished by putting in a key, or by the use of $\frac{3}{8}''$ diameter bars about $2'-6''$ long set at an angle of 45 degrees with the horizontal, having half of the length of each bar on each side of the joint. These bars can generally be inclined in one direction only, since the shear, if large enough to be of importance, will hardly ever reverse.

THE DESIGNING OF COLUMNS

Columns are to be figured for direct vertical loads, and also for the bending stresses induced in them by eccentric loads, by the deflections of girders which are continuous with them, and by temperature changes, wind loads, and traction loads. The methods of calculating the moments produced by these different causes have already been explained under the heading "The Calculation of Stresses in Monolithic Structures"; and the resulting stresses in the concrete and steel therein can be computed by means of Figs. 37*k*, 37*m*, 37*o*, and 37*q*. The "Specifications for Design" in this chapter give the unit stresses to be used with the various combinations of loadings.

Columns without hooping are generally rectangular in section. The main reinforcement consists of bars near the outside of the section, which bars must be very effectively secured against buckling outward, at points spaced not to exceed fifteen (15) diameters of the bar. The reinforcement must be arranged so that a concrete chute can be used; and it is desirable, in large columns, that sufficient space be left to permit the passage of a man. Mr. Howard's paper on the Twelfth Street Trafficway at Kansas City shows details for columns of the type just discussed.

Hooped columns should be either round or octagonal, since the hoops must be round. The "Specifications for Design" indicate the amount and spacing of hooping required, and the permissible percentages of longitudinal steel.

Splices in the main bars should usually be made by lapping them the proper amounts. At sections where there is no tension, the bars are sometimes butted, the two ends being faced and held in line by a sleeve-nut or a tight-fitting sleeve.

The details of columns at expansion joints must be arranged to suit the longitudinal girders and the cross-girders. In some cases the cross-girder and one main girder are continuous with the column, and the other main girder slides; while in other instances both main girders are free to move, the cross-girder being carried directly on the column. This latter arrangement permits the distance between expansion columns to be one

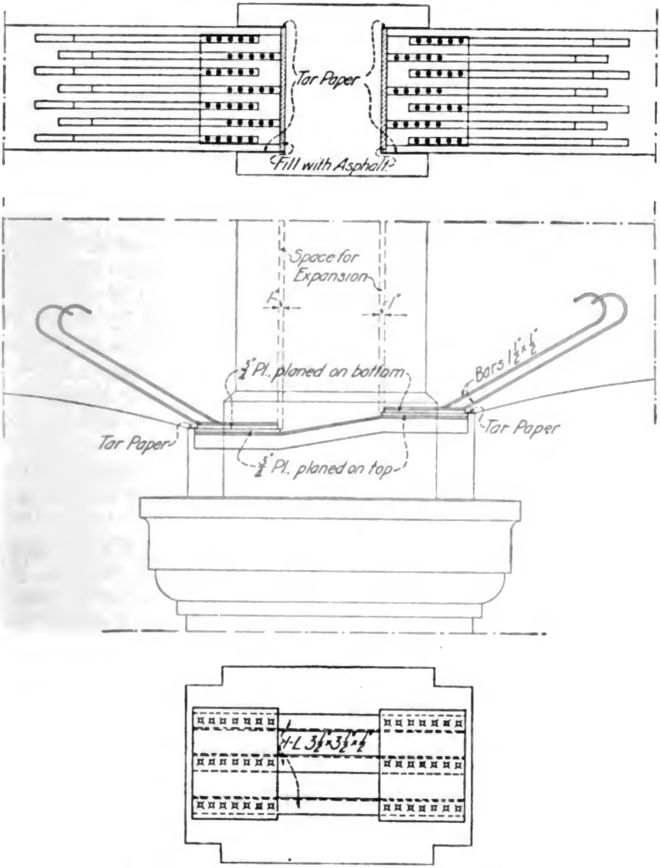


FIG. 37*uu*. Expansion Plates for Main Girders.

span longer than does the former; but it increases the total number of expansion joints, since there are then two at each expansion column. Especial care is necessary in either type to prevent the plates on which the girder-shoes slide from being torn loose from the column, and to prevent the column from being split. On the Twelfth Street Trafficway, where both girders were arranged so as to slide on the expansion columns, the plates under the two girder shoes were connected to each other by angles, as shown in Fig. 37*uu*. When only one girder slides on the column, similar angles should be used, carried well over into the concrete of the girder which is continuous with the column.

The architectural treatment of the columns should be in conformity with the lines of the remainder of the structure. For plain, massive work, in which there is no ornamentation, rectangular columns with vertical sides will prove quite satisfactory. For more elaborate structures, it will be best to use capitals and plinths, and to batter the sides of the shaft. A batter of one-eighth (⅛) of an inch to the foot is about right for this purpose. For large columns, the use of panels on the sides of the shaft is advisable. These panels may be either raised or sunken, to suit the taste of the designer. For a double-deck structure, the use of pilasters under the main girders of the lower deck adds to the appearance of the structure. As was just mentioned, Mr. Howard's paper shows the type of column adopted for the Twelfth Street Trafficway, which was a double-deck structure.

When capitals are employed, some form of reinforcement should be adopted to prevent their cracking off. Either small bars or wire mesh can be used for this purpose.

Construction joints require little attention on the part of the designer, and may be located at any desired section. There will always be one such joint at the top of the footing, and generally one at the bottom of the girders. Since it is usually inconvenient to place the column reinforcement before the footing is poured, the main reinforcement should begin at the top of the footing, and an equal number of short bars or dowels should be placed in the latter, extending far enough into it and into the column shaft to be developed fully in both.

THE DESIGNING OF COLUMN FOOTINGS

The formulæ given on page 857 *et seq.*, in the section of this chapter entitled "Stresses in Column Footings," will enable the stresses in such slabs to be computed without difficulty. Fig. 37*b* can be utilized for the design of the section after the moment has been calculated.

The thickness of a footing must be tested for the bending moment on the section at the edge of the column shaft, for punching shear on this same section, and for diagonal tension on the section distance $\frac{d}{2}$

from the edge of the shaft. The footing should be made of such thickness that web reinforcement will not be required.

The unit bond stress is to be computed by Equation 126. It will usually be found necessary to adopt rather small bars, in order to keep the bond stress down to a proper value; and testing for this stress should never be omitted. All bars should extend to the edges of the footing, since any that are stopped off will not be properly developed.

The tops of footings should be kept horizontal. Footings with stepped upper surfaces did not show up well in Prof. Talbot's tests reported in Bulletin No. 67 of the Engineering Experiment Station of the University of Illinois; and sloping the top surface causes extra form work and makes the placing of concrete more difficult, which about offsets the saving due to the decrease in the amount of concrete required.

The reinforcement should be placed at least three (3) inches from the bottom of the footing. As was stated previously, the steel area per foot width should be determined by Equations 159 and 160; and the spacing of the bars thus found must be kept constant over the entire width of the footing. The two sets of bars should be securely wired at their intersections. Dowels extending up into the column shaft will also be required, as discussed under "The Designing of Columns." They should be put into position and securely fastened before the pouring of concrete is begun.

In designing plain footings for bending moments, a fairly high value of the working modulus can be used. If the moduli of rupture for the footings tested by Prof. Talbot be figured for the moments per foot of width given by Equations 159 and 160, the values will be found to range from 280 to 500 pounds per square inch, with only two values less than 350 pounds per square inch. These footings were of 1:2½:5 concrete. Apparently, a working modulus of 60 pounds per square inch can be safely used for 1 : 3 : 5 concrete, and one of 70 pounds per square inch for 1 : 2 : 4 concrete.

THE DESIGNING OF WALL FOOTINGS

The locations of the critical sections for bending moment and diagonal tension have been discussed in the section entitled "The Calculation of Stresses in Wall Footings," and formulæ have been given for calculating the moments and shears. If the top of the footing is horizontal, Fig. 37*b* is to be employed in the design of the section for bending moment, and Equations 125 and 126 in figuring the unit shearing and unit bond stresses. If the top surface is inclined, Fig. 37*j* is to be used in designing the section for moment; and Equations 139, 125, and 126 in figuring the unit shearing and unit bond stresses.

Footings should be made of such thickness that shear reinforcement is unnecessary. The unit bond stresses should always be investigated, as they are likely to be high. All bars should extend to the edge of the footing.

The statements made in the preceding section of this chapter regarding the use of sloped or stepped tops for column footings apply also to wall footings, although the employment of sloped tops is occasionally desirable.

In designing plain footings for bending moment, a rather high working modulus can be used, as will be seen from Prof. Talbot's tests referred to on the preceding page. A working stress of 60 pounds per square inch would appear to be satisfactory for 1 : 3 : 5 concrete, and one of 70 pounds per square inch for 1 : 2 : 4 concrete.

THE DESIGNING OF ARCH SPANS AND PIERS

Reinforced-concrete arch spans are of two general types, solid-spandrel and open-spandrel. In the first form the arch-ring must be solid, and the roadway rests on a fill which is carried on the said arch-ring and is restrained transversely by spandrel-walls along the edges thereof. In the second form, the arch-ring may be of either the solid-barrel or the ribbed type. The roadway is supported on reinforced concrete slabs, which are carried by cross girders and spandrel columns when the ring is of the ribbed type, and by transverse spandrel walls when the ring is solid. The solid-spandrel and open-spandrel forms are frequently combined in one span, the centre portion being of the former type and the end portions of the latter.

The solid-spandrel arch, which is frequently termed the spandrel-filled arch, is best adapted to spans of low rise, as the weight of the filling becomes excessive in an arch of high rise. Also, the spandrel wall, which acts as a retaining wall, concentrates a heavy load along the edges of the ring when it is of considerable height. However, it is occasionally used for railway spans of high rise, in order that the weight of the filling may absorb the impact of moving trains. The arch ring should, preferably, extend the full width of the roadway, as the best looking structure is secured thereby. A considerable saving in the arch ring and substructure can be effected by making the arch ring narrower than the roadway; and when the appearance of a bridge is not of vital importance, the economy may be justified. The outer portions of the roadway are then carried on cantilever brackets which are supported by the spandrel walls and are continuous with cross-walls resting on the arch ring or with cross-struts.

The open-spandrel arch is more economical than the solid-spandrel one for all spans except those of very low rise. The ring should be solid when the ratio of the rise to the length is small, and also when the high water line is much above the springing; but under other conditions the ribbed type will be preferable. Since the roadway and the arch-ring are entirely separate members of the structure, there is no loss in appearance if the latter is narrower than the former; and the economy resulting from this arrangement should always be realized. The overhanging portions of the roadway will, of course, be supported on cantilever beams continuous with the cross-girders or transverse spandrel-walls. Longitudinal spandrel-girders, placed under the floor-slabs with their outer faces about in line with the outer faces of the arch-rings, will add somewhat to the appearance of the span. Their under surfaces should be arched, to be in harmony with the type of structure. When the ribbed type of arch ring is used, the ribs must be braced by efficient cross-struts, spaced not to exceed twelve times the width of the ribs.

The hingeless type of rib has generally been employed in the United States, but three-hinged arches have been built extensively in Europe. The great advantages of the hinged rib are that the stresses are determinate, that the amount of calculation work required is comparatively small, and that the temperature stresses are eliminated; while its chief drawback is its somewhat awkward appearance, due to the fact that the thickness in the haunches is greater than that at the springing, unless a considerable amount of concrete is used which is not needed so far as the stresses are concerned. The appearance of the hingeless rib is very fine. The stresses are to some extent indeterminate, but experience has shown that the ribs are perfectly safe in every way when designed in accordance with the elastic theory; and the calculations required are not unreasonably long. The temperature stresses are high in flat arches, as are also the stresses resulting from rib shortening and from small movements of the supports. The hingeless rib is stiffer than the other; but this is not of great importance in concrete structures, since the dead load is a large proportion of the total. The cost of the two forms is not materially different except for flat arches, under which circumstances the three-hinged type is somewhat cheaper. That type requires less concrete than the other, but the cost of the hinges will about offset this saving in other than flat arches. When the bridge over the Arroyo Seco at Pasadena, Cal., was designed by the author's firm, complete estimates were made for both the hingeless and the three-hinged types. The hingeless type was found to be a trifle cheaper than a three-hinged rib of economic outline, and decidedly cheaper than a three-hinged rib in which the thickness at the springing was increased for the sake of appearance. The arches in this case were of high rise, as can be seen in Fig. 52*f*, which is a reproduction of a photograph of that bridge. An extended discussion of the relative merits of the two types of ribs is to be found in a paper by W. M. Smith, Sr., M. Am. Soc. C.E., and W. M. Smith, Jr., Jun. Am. Soc. C. E., entitled "Concrete Bridges: Some Important Features in Their Design," in Vol. LXXVII of the *Trans.* Am. Soc. C. E., and the accompanying discussions.

The character of the substructure required will vary widely in different arch structures. When there are several consecutive spans, the intermediate piers, below the springing lines, will often not differ essentially in type from those used for steel bridges, although it will frequently be necessary to reinforce the shafts and to employ unusually wide bases; and special care must be taken to secure unyielding foundations. In long bridges of this type, wide abutment-piers, having sufficient stability to stand if one of the arch spans that it carried were removed, should be placed at intervals. This will prevent the entire structure from being wrecked if one span or pier should fail. The end supports are entirely different from those used for ordinary steel bridges, since they have to care for a large horizontal thrust. The front face of the arch abutment proper is usually made vertical, while the rear face has a very large batter. A rather unusual condition arose in the case of the Fifteenth Street Bridge over the Blue River at Kansas City, Mo., which was designed by the author's firm. In this case the shale on which the abutment was to rest was overlaid by about forty feet of soft clay. A rectangular reinforced concrete crib was constructed and sunk by the open-dredging process to the desired depth. The arch ring rested on top of this crib, near the front edge.

The treatment of the intermediate piers above the arch springings is a matter of æsthetics. The upper portion of the shaft should be wide, preferably nearly as wide as the shaft below the springing; and if cantilever beams are used, there should be one nearly the full width of the upper portion of the shaft, or else two, one at each edge thereof. The end pier should be made similar to the intermediate ones, if short approach spans are adopted. When an abutment is required in order to retain an earth fill, it should usually be made of the U-type; and it should generally extend the full width of the roadway, even though the arch ring is narrower. In the case of open-spandrel arches, it will be found somewhat difficult to make the treatment of the abutment harmonious with that used for the upper portions of the intermediate piers.

In the Twelfth Street Trafficway it was found necessary to use an arch of 134′ span, the springing of which had to be placed a considerable distance above the ground level. As there were girder spans at each end of the arch, it would have required expensive abutments to resist the horizontal thrusts; and, furthermore, there was no space available for such abutments. The difficulty was overcome by using a bottom chord of steel eye-bars to take up the said thrusts. A complete description of this arch, with full details, is to be found in the paper by Mr. Howard in the *Proceedings* Am. Soc. C. E. for May, 1915, which has been mentioned previously. Attention is called especially to the rockers for taking care of expansion, and to the toggle in the eye-bar chain, which was used to eliminate the stresses in the arch ring due to the stretch of the tie and to the shortening of the arch rib; and also to the fact that the arch rib is free from temperature stresses.

In the open-spandrel type of arch, the only expansion joints required are in the floor-system and handrails. For short spans these joints should be located at both ends of each span; and for longer spans there should also be provision for expansion at about the quarter points. In the solid-spandrel type there should be expansion joints in the spandrel walls, floor-system, and handrails at each end of each span; and in long spans there should also be expansion joints at the quarter points.

The question of æsthetics should be carefully considered in designing a reinforced-concrete arch span. Chapter LII discusses this subject. Fig. 52*e* shows an example of an arch bridge with open spandrels in which the lines are very plain, and Fig. 52*f* illustrates a structure of the same type which is somewhat ornamental. Fig. 52*h* shows an arch bridge with closed spandrels. Typical details of the bridge illustrated in Fig. 52*f* are to be found in an article on page 146 of the *Engineering News* for July 24, 1913, and in Part III of Hool's "Reinforced Concrete Construction."

The design of the floor-system of an arch span involves no peculiar features. The spandrel girders, if such are used, will be made continuous between expansion joints. The design of the spandrel columns is also simple. Their appearance is improved by battering them slightly and using capitals and plinths: but their treatment must be made consistent with that of the piers. A good example of details of this nature is to be found in the bridge shown in Fig. 52*f*, reference to which was made in the preceding paragraph.

The methods of laying out the arch ribs and calculating the stresses therein have already been fully treated in this chapter. The main reinforcement should consist of bars placed near the intrados and the extrados. At least one-half of one per cent of the area at the crown should be used in each face throughout the rib, and extra steel should be added at the critical sections as required by the stresses. In arches of high rise, additional bars will be needed in the haunches, and possibly at the crown; while in flat arches it will be necessary to add a large amount of steel at the springing, and usually a smaller amount at the crown. The percentage of steel required in any case will depend partly upon the ratio of the live load to the dead load, increasing as this ratio increases. The bars in each face should, preferably, be arranged in one row, with their centres at least 3 inches from the edge of the concrete; but two rows will frequently be required at the critical sections. Bars should be properly lapped at all splices. Since the concrete in the piers or abutments up to the springing line will generally be poured before the forms for the arch ribs are constructed, it will be necessary to splice all bars there; but not more than half of the bars should be spliced at any other point.

The transverse reinforcement in a solid-barrel arch rib should consist of bars $\frac{1}{2}''$ or $\frac{3}{8}''$ in diameter, spaced about two (2) feet centres in each face. These bars should be securely wired to the main bars at each intersection. Stirrups tying the top and bottom steel together should also be used, in order that the steel near the extrados may not buckle outwards. The stirrups should hold each main bar at its intersection with each transverse bar. Continuous bars, arranged like the web-members of a Warren or triangular truss, are preferable to separate loops for these stirrups.

Each rib in an arch of the ribbed type should be hooped or banded in about the same manner as a column. In a deep rib it will be necessary to place longitudinal bars in each vertical face; for the rib is a column transversely. However, it is practically free from the secondary stresses present in most columns. Examples of ribs of this type are to be found in the articles previously referred to in the *Engineering News* of July 24, 1913, and on page 1019 of the *Proc.* Am. Soc. C. E. for May, 1915.

The cross-struts used to brace the ribs in a span of the ribbed type should be figured for wind stresses and for their own weight. The dimension parallel to the axis of the rib should be about equal to the width of the rib, in order that the strut may be quite stiff. The spacing of the struts should be about eight (8) times the width of the ring. Usually a strut at the crown is unnecessary, as the cross girders near this point will afford proper transverse support in most instances.

The design of the upper portion of a pier between two arch spans is simple. The method of calculating the stresses in the shafts below the springings and the pressures on the foundations of such piers has already been explained. When a pier carries two spans that are unlike, the bending moments at some sections of the shaft and at the base are apt to be high. These moments can frequently be reduced by shifting the arch ribs slightly. The springing of the arch having the larger horizontal thrust should be placed below that for the other arch, so that the resultant of the reactions from the two arches may strike as near to the centre of the footing as possible. In some cases it will be economical to make the footing unsymmetrical about the centre line of the shaft. In adopting these economic expedients, however, care should be taken not to violate the principles of æsthetics.

Fig. 37*ll* shows the section used for the shaft of a pier between two equal arches, and also the reinforcement employed. Details of piers used in two of the author's bridges are to be found in the articles in the *Engineering News* and the *Proc.* Am. Soc. C. E. just referred to.

The calculation of the stresses produced by arches in their abutments has also been previously discussed. If an abutment acts also as a retaining wall, the effects of the earth pressure must be properly combined with those from the arch rib. Fig. 37*kk* shows the outlines and reinforcement used for the abutment of an arch span in which the springing was located very close to rock. The footing was carried into the rock for some distance in order to secure the requisite resistance against the horizontal thrust of the arch. This point should never be overlooked when arch abutments are being designed.

HANDRAILS

Concrete handrails are of two general types: solid and open.

The solid handrail should be used on massive structures only, for it gives a top-heavy appearance to lighter ones. In the Twelfth Street Trafficway at Kansas City, Mo., half-through girders were used for the lower deck, and the upper portion of these girders was paneled to resemble a solid handrail. As has been previously stated, this structure is described fully in a paper in the *Proc.* Am. Soc. C. E. for May, 1915; and the appearance of the railing can be seen in Plate XV and Fig. 22 of that paper. A handrail of similar outline was used on a small arch bridge designed by the author's firm several years ago, but the appearance was unsatisfactory.

Open handrails are of two forms. In one kind each panel consists of a thin slab with open spaces forming a design of some sort, such as a Greek cross; while in the other vertical balusters are used. The author has employed the former type on several small bridges. The latter type was adopted for the upper deck of the Twelfth Street Trafficway. The details of the handrail on this structure are given in Plate XVII of the paper previously mentioned, and its appearance can be seen in Figs. 21, 23, 24, and 25. It will be noted that the balusters are plain. In the Arroyo Seco Bridge at Pasadena, Cal., a view of which is shown in Fig. 52*f*, the same type of railing was used; but in this case the balusters were quite ornamental, the top and bottom portions being square, and the centre portions round and swelled out, thus giving to the baluster an urn-like appearance.

The handrail should be so detailed as to accentuate the prominent features of the structure beneath. Small posts should be used at each cross girder or cantilever, and very prominent ones at each pier or column. In the Arroyo Seco Bridge, small bays were formed over each pier; and similar bays were adopted at the ends of the arch span in the Twelfth Street Trafficway, as can be seen by referring to Plate XIV and Figs. 14 and 21 of Mr. Howard's paper. In the latter case these bays help to emphasize the prominence of the arch span over the rest of the structure, being thoroughly in keeping with the very massive arch piers.

The treatment of the handrails should harmonize with that of the remainder of the structure. Thus the lines of the Twelfth Street Trafficway are simpler than those of the Arroyo Seco Bridge, and the structure is much more massive, so that plain, substantial balusters were used rather than the more ornamental ones employed on the California Bridge.

Handrails should be designed so that the balusters, the top rails, and the caps of the posts can be separately moulded, as better form-work will be obtained thereby. The bottom rail and the posts are usually poured in place; but they can be precast, if desired.

The bottom rail is frequently raised an inch or more above the top of the sidewalk slab, in order to provide a space for the removal of dirt, thus preventing its accumulation on the sidewalk.

Enough reinforcement should be used in the various portions of a handrail to prevent cracking; however, nothing but a few ¼″ or ⅜″ bars should be employed, as otherwise the placing of concrete would be very difficult. Frequently provision must be made in the top rails for conduits for light-wires; and the posts will often have to support lamp-posts and sometimes trolley poles. Lamp-posts, when employed, should be of an ornamental nature.

CONSTRUCTION WORK

The construction of reinforced concrete bridges requires very careful supervision on the part of the engineer, as faulty work may make a well-designed structure unsafe. For the best results the engineer who makes the design should supervise the construction.

The 1913 Report of the Joint Committee covers in a general way the most important features of construction work, and the portion of that report relating thereto should be carefully studied.

The materials to be used in the making of concrete should be thoroughly tested. The cement must be Portland, no other kind being permissible. The testing of the cement is usually performed in a satisfactory manner; but the testing of the fine and coarse aggregates is frequently neglected to a serious extent, although it is of vital importance.

Compression tests should be made on 8″ × 16″ cylinders of concrete, as provided in the Specifications of Chapter LXXIX. The making of these tests is a very necessary part of the field inspection, for it is the only means of determining whether the strength assumed in the design is actually attained in the structure. As is a matter of common knowledge, the strength of concrete frequently drops much below the values which are generally assumed as standard. The strength specified for the concrete of any particular structure should be such as the engineer knows can be secured by the use of aggregates obtainable in that locality; and he should insist upon these requirements being met, even though the contractor be compelled to use a richer mixture. The specifications should cover this point fully.

The falsework must be of adequate strength. Some initial settlement will necessarily occur, and this should be allowed for by cambering the forms; but any subsequent settlement is likely to endanger the strength of the structure. It will be well to place wedges between the falsework and the forms, to take care of unusual settlement at any point. Generally the contractor should design the falsework; but the engineer should check the plans thereof thoroughly before permitting construction work to proceed. The falsework for arches and for girders high above the ground will require specially careful consideration. Timber falsework is ordinarily employed, yellow pine or Douglas fir being the best kinds of wood for the purpose. Steel centering will occasionally be necessary for arches, and for spans where falsework resting on the ground cannot be used.

The first requirement for the forms is that they shall be so strong that they will not break, or even bulge perceptibly. The pressures for which they should be designed are not known definitely. A number of experiments have been made, but the results are not in close agreement. Articles on the subject have appeared in the *Engineering News* as follows: September 9, 1909, page 288; June 30, 1910, page 748; July 28, 1910, page 103; August 17, 1911, page 211, and August 14, 1913, page 294. There are also articles in the *Engineering Record* of January 15, 1910, page 71, and July 20, 1912, page 84. Some of the writers of these articles suggest that forms should be designed for the pressure exerted by a fluid weighing 80 or 85 pounds per cubic foot, and this value has been used to some extent. Taylor and Thompson, in their book entitled "Concrete Costs" (1912 edition), give tables for the design of forms on page 607 *et seq.* They adopt as a basis the results obtained by Major F. R. Shunk in 1908, and reported in the preceding references to *Engineering News* of September 9, 1909, and to *Engineering Record* of January 15, 1910. Major Shunk's tests were made on very wet concrete, and took into account the effects of variations in temperature and in the rate at which the forms were filled up. They probably afford the best data on the subject at present obtainable.

Fig. 37*vv* presents the results of these tests in a form somewhat better suited for direct use than that given in "Concrete Costs," as it enables the unit pressure at any distance below the top of the concrete to be determined. The dotted curves were drawn in by extending Major Shunk's diagram, and they probably give pressures that are too great. The full curve extending nearly to the top of each diagram was derived from the tests reported by the Aberthaw Construction Co. in *Engineering News* of August 14, 1913. These tests were made on two hooped columns twenty (20) feet high and twenty (20) inches square in a building under construction, with ordinary working conditions, the concrete being fairly wet. One column was filled in 9 minutes and the other one in 14 minutes, so that the rates of filling were about 130 feet and 85 feet per hour, respectively. The results from the two cases checked closely, and indicated nearly full fluid pressure at a point eighteen (18) feet below the tops of the columns.

The curves of Fig. 37*vv* are drawn for temperatures of 40 degrees, 60 degrees, and 80 degrees. For intermediate temperatures, the pressures can be interpolated. Major Shunk's article gives no data for temperatures above 80 degrees or below 40 degrees. It is probable that for temperatures exceeding 80 degrees there is no great reduction in pressure, but a considerable increase for temperatures below 40 degrees. For a temperature just above 32 degrees the setting would proceed so slowly that nearly full fluid pressure could be expected, unless the rate of filling were very slow.

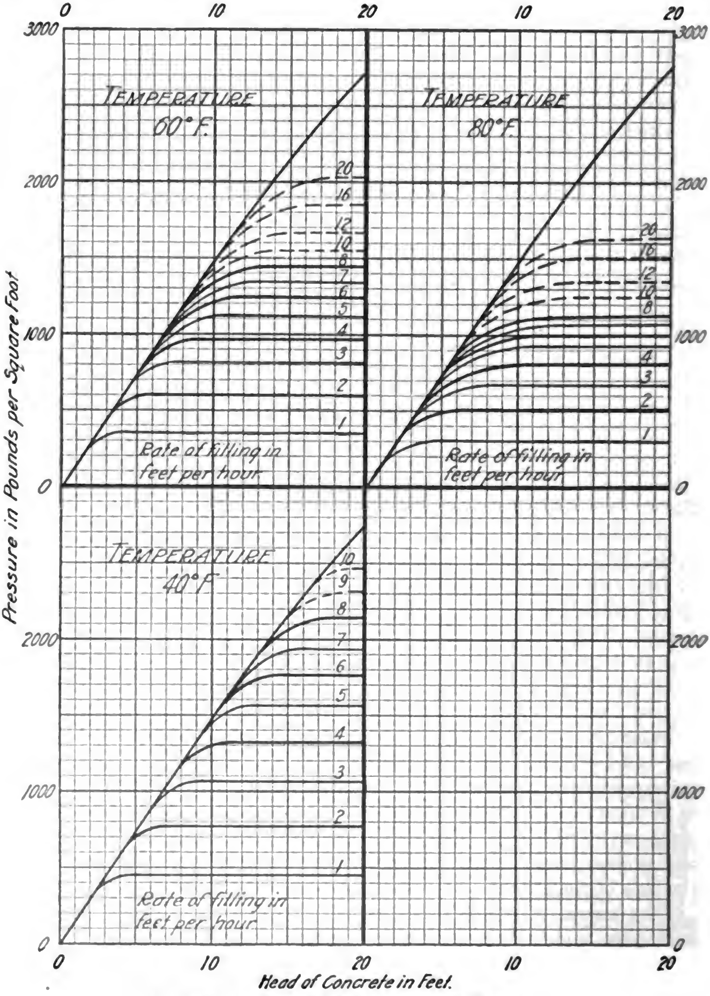


FIG. 37*vv*. Pressure of Wet Concrete on Forms.

Major Shunk's tests were criticized in some of the articles above referred to, it being pointed out that forms designed for the pressure of a fluid weighing eighty (80) or eighty-five (85) pounds per cubic foot have given satisfaction in many instances. It will be noted, however, that Major Shunk's tests indicate values but little greater than these for usual working conditions, so that the above objection carries no weight. Major Shunk's curves take into account the main factors which enter into the problem; and their use is, therefore, to be preferred to the other method, which makes the strength of the forms independent of the rate of pouring and of the temperature.

After the unit pressure on the forms has been determined, they must be tested for both strength and stiffness. The tables previously referred to on page 607 *et seq.* of Taylor and Thompson's "Concrete Costs" can be used to advantage for this purpose. The values given therein limit the unit stress to 1,200 pounds per square inch, and the deflection to $\frac{1}{8}''$, which amounts are entirely satisfactory. Fig. 16*h* in this treatise can also be utilized for the design of forms.

Especial care must be given to the designing of forms for columns and thin walls in which the concrete will be filled up rapidly.

Forms should ordinarily be constructed of yellow pine or Douglas fir. White pine is best for nice work and where a good deal of framing is required. Metal forms can be employed to advantage when they can be used a great many times, as in handrails, column capitals, and ornamental details. The sheathing for slab forms and side forms of girders should be 1″ thick. Stock $1\frac{1}{2}''$ thick is best for columns, and 2″ thick for arch ribs and the bottom forms of girders.

The two sides of a form should be secured to each other by bolts or wire. The use of shores is not so satisfactory, since they are likely to settle. Bolts should be well greased or enclosed in tin or pasteboard tubes, so that they can be withdrawn after the concrete has set. The holes should then be filled up with grout. Cutting off the bolts near the surface of the concrete is not advisable; but if for any reason it should be necessary, they should be cut off at least one inch back from the face of the concrete.

Forms must be tight enough to prevent leakage. The use of tongued-and-grooved lumber or ship-lap is good from this standpoint. Auger-holes or large cracks should be plugged with timber, and the surfaces then planed. Small cracks can be plugged with very stiff clay. Wetting the forms thoroughly a short time before concreting begins will swell the timber and close up small cracks.

The forms for surfaces exposed to view must be constructed of lumber of uniform thickness, dressed on at least the side next to the concrete. The use of tongued-and-grooved stock will give the best results; and ship-lap is better than plain lumber. The latter can be employed when the surface of the concrete is to be treated after the forms are removed, or when the appearance is not of great importance; but in other cases ship-lap or tongued-and-grooved stock should be adopted. Plain lumber must be dressed on the edges. The specifications for any structure should state clearly the kind of form-lumber that will be required. Bevel strips should ordinarily be used at all sharp corners of the forms—either re-entrant or salient.

In constructing forms at expansion joints, care must be taken to see that all portions can be removed after the concrete is poured. Vertical boards employed to form the expansion openings in slabs should be wrapped in tar-paper, and, preferably, tapered slightly. For deep expansion openings at the ends of large girders, a metal form is best. On the Twelfth Street Trafficway at Kansas City, Mo., these openings were over eleven (11) feet deep. The contractor used 1½-inch square bars, well greased. They were withdrawn the day after the concrete was poured, as it was found very difficult to get them out if they were left in place for a longer time. Care must be taken to see that expansion openings are not allowed to fill up with dirt before the specified filling—usually asphalt—is put in place.

Bulkheads for construction joints in slabs and girders should be vertical. In slabs they are best made of boards running horizontally and notched down over the reinforcing bars. For girders, boards running vertically will generally be used. Keys can be formed by blocks or boxes nailed to the bulkhead; and these should be tapered so that they can be easily withdrawn. The keys must be arranged so as to prevent both horizontal and vertical displacement. They must not be too large, no dimension exceeding one-third of the corresponding dimension of the girder being permissible. At horizontal construction joints, similar keys can be constructed, or else large stones can be used for this purpose.

Construction joints should be located only at such points as the engineer may designate.

The engineer must check over the forms carefully after they are constructed, to see that all dimensions are correct.

The contractor should submit to the engineer complete working drawings showing the arrangement of the reinforcing bars and the location of the splices; and these should be checked carefully. The reinforcement should be thoroughly inspected after it is in place, to see that the plans have been adhered to in every respect, and that the bars are securely fastened against displacement. The surfaces of the bars should be free from grease, dirt, oil, paint, or scaly rust. A small amount of rust which is neither scaly nor flaky is not objectionable.

The bottom reinforcement of slabs should be supported by special clips or by small cement blocks. The former are better, as the blocks frequently crumble. The supports should be spaced so closely that the bars will not bend down appreciably under the weight of a man. Bent-up bars, when used, will generally support the top steel well enough; otherwise some form of spider must be adopted for this purpose. Placing the top steel after the concrete has been poured should not be permitted.

Bars should be securely wired or clipped to each other at all intersections.

The bottom reinforcement of girders can be supported by stirrups, by concrete blocks, or by form bolts as shown on page 501 of Taylor and Thompson's "Concrete Costs" (1912 edition). Where more than one layer of bars is used, each upper layer should be supported on short pieces of pipe resting on the layer of bars beneath, as shown in the detail last referred to. The top steel will be supported to some extent by the bent-up bars and by the slab reinforcement; and additional support can be provided by suspending the bars by wire from timbers resting on the forms. The main bars, cross bars, and stirrups should be securely wired to each other at all intersections.

The steel in arch ribs can be supported in much the same manner as that in girders. The top reinforcement can be carried by cross-timbers resting on the forms.

The steel in column footings can best be supported on concrete blocks. The bars should be wired at their intersections.

The longitudinal bars in columns should be wired to the bands or hoops to form a unit frame. This frame must then be secured against displacement.

Expansion shoes and plates should be placed to exact position, and the sliding surfaces thoroughly coated with a thick grease containing a lubricant such as graphite or mica. All surfaces of the concrete in contact at expansion joints must have two layers of tar paper between them.

The mixing of concrete requires careful supervision at all times. Enough water should be used to ensure that the concrete will work around the reinforcement properly; but the amount should be kept as small as is consistent with this result, because an excess of water lowers the strength of the concrete considerably. Each batch must be kept in the mixer until it is thoroughly mixed. Especial attention should be given to this point when thin walls or other small sections are being poured.

Concreting in freezing weather is rarely advisable. It should be permitted only when the constituent materials are heated well above the freezing point, and when provision is made to keep the concrete from freezing until after it has thoroughly set. The use of salt is not advisable, as it increases the danger of electrolytic action.

The placing of concrete requires constant inspection. Forms must always be cleaned out and thoroughly wet before the concreting begins; and where fresh concrete is to be placed against old concrete, all *laitance* should be removed therefrom, and the old surface wetted and covered with a thin grout consisting of one part of cement and two parts of sand. If the old concrete has stood for several days, it will be best to roughen its surface with a pick before applying the grout.

The concrete should be transferred from the mixer to the forms as quickly as possible, and in such a manner that no segregation of the materials will occur. It should not be allowed to drop into the forms from any considerable height—never more than eight (8) feet, and preferably not more than two (2) feet. The use of chutes or trémies is always advisable; and for high walls or columns they are necessary. The concrete should be worked around the reinforcement thoroughly; and it should be well spaded, especially next to the forms. The bars in the bottoms of slabs should be picked up slightly and shaken by means of pick-up bars; and all other bars should be shaken, particularly in columns. At construction joints, the concrete should not be spaded, but the surfaces should be left rough. The formation of *laitance* should be prevented as far as possible, and any that occurs should be removed before it has time to harden.

In order to be certain that slabs are poured to the proper thickness, screeds of some sort should be used, on the top of which templates can be placed. These screeds should be removed as soon as the concreting is finished.

The method of concreting an arch rib will depend upon its size. For small spans, it will be satisfactory to pour consecutively from both abutments to the centre, making construction joints if necessary; but for a large span, such a procedure is likely to cause the falsework to settle unevenly, and to produce cracks in the concrete first poured. For such ribs it is best to divide the arch up into several sections by means of bulkheads placed at right angles to the axis of the rib. Alternate sections are then poured in such order as to keep the load on the falsework somewhat uniform. The remaining sections are then concreted, the one at the crown being the last. Descriptions of construction work of this nature can be found in a paper on the 233-foot arch span of the Walnut Lane Bridge at Philadelphia in Vol. LXV of the *Trans.* Am. Soc. C. E., and in an article in *Engineering Record* of January 2, 1910, on the 280-foot Rocky River arch at Cleveland, Ohio. In the case of a solid-barrel arch, the rib can be divided into two or more longitudinal strips by bulkheads, and the various strips concreted separately.

After concrete has been poured, workmen should not be permitted to walk on it for at least twenty-four (24) hours. In hot weather, slabs should be thoroughly wetted two or three times a day for a week after pouring. Workmen should not be allowed to build fires on slabs.

The proper time for the removal of forms is a more or less uncertain matter, as there are many variable factors entering into it. In general, it may be said that forms can be removed from vertical members more quickly than from horizontal ones, from slabs and girders of short span more quickly than from those of longer span, and more quickly in hot weather than in cold weather. If concrete becomes frozen while green, the forms should be left on for a long time after it has thawed out, as the hardening process is likely to proceed very slowly. In the author's practice, slab forms and the side forms of girders have been left on about one week in warm weather, and two weeks in cold weather. The falsework under ordinary girders has been kept in place about two weeks in warm weather, and in cold weather a month or more; but for long, heavy spans a somewhat greater time has been allowed. The forms for small columns have been left on for a few days in warm weather, and for a week in cold weather; while for large, tall columns, they have been removed in about the same time as have slab forms. The falsework for small arches has been left in place about as long as that for ordinary girders; but for long, heavy arches it has not been removed until after considerably more time has elapsed.

The periods given above are to be regarded as merely approximate, and should not be used blindly. The rate at which concrete attains its strength will be found to vary considerably on different jobs, even when the temperatures are the same, due to differences in both the cement and the aggregates, and also in the mixing and placing. In particular, an excess of water will retard the hardening process considerably.

The hardness of concrete can be told to some extent by striking it with a hammer. If this gives a clear ring, the concrete is probably pretty well set up.

The most satisfactory method of determining the proper time for the removal of forms is to make test specimens in addition to those for the standard tests, keep them under the same conditions as the concrete in the structure, and then have them broken at the time it is proposed to remove the forms. This method was followed on the Rocky River Bridge mentioned previously. A few such tests, made in connection with the more important members of the structure, will prove sufficient, unless the results are erratic.

The surface finish of reinforced concrete work is very important. A good discussion of this question is to be found in the 1915 Report of Committee on Masonry of the American Railway Engineering Association. (See *Proceedings* for 1915, page 800.)

SPECIFICATIONS FOR DESIGN

1. *Classification of Bridges.*

As regards these specifications, all structures will be divided into three general classes: steam railway bridges, electric railway bridges, and highway bridges. Highway bridges will be divided into three classes, viz.: Class *A*, which includes those that are subject to the continued application of heavy loads; Class *B*, which includes those that are subject to the occasional application of heavy loads; and Class *C*, which includes those for ordinary, light traffic. In general it may be stated that bridges of Class *A* are for densely populated cities, those of Class *B* for smaller cities and manufacturing districts, and those of Class *C* for country roads.

Elevated railway structures will be considered as electric railway structures.

Loads

2. *Loads for Steam and Electric Railway Bridges.*

Steam railway and electric railway bridges are to be designed to sustain properly the stresses produced in them by any of the following loads, or by any combination of them which may reasonably be expected to occur:

A. Live Load.
B. Impact load.
C. Dead load.
D. Effects of arch shortening (for arches only).
E. Wind load.
F. Traction load.
G. Centrifugal load.
H. Effects of changes of temperature.

3. *Loads for Highway Bridges.*

The loads to be considered in designing highway bridges are as follows; and all parts of such structures are to be designed to sustain properly the greatest stresses produced therein for all reasonable combinations of the various loadings, excepting only that the live load and the wind load are not to be assumed to act simultaneously:

A. Live load.
B. Impact load.
C. Dead load.
D. Effects of arch shortening (for arches only).
E. Wind load.
F. Effects of changes of temperature.

4. *Live Loads for Steam and Electric Railway Bridges.*

The live loads to be used in designing any steam railway structure shall be taken from Figs. 6*b*, 6*c*, 6*d*, and 6*e*, and those for designing any electric railway structure, from Figs. 6*f* to 6*n*, inclusive. The equivalent live loads are to be used in making stress computations instead of the actual wheel concentrations.

In case electric railway tracks are carried on bridges which also accommodate highway traffic, the live load on each track is to be assumed to occupy a space ten (10) feet wide to the exclusion of other live loads.

5. *Live Loads for Highway Bridges.*

The uniformly distributed live loads per square foot of floor, including the entire clear widths of both main roadways and footwalks, shall be taken from the curve diagram shown in Fig. 6*o*; and the concentrated live loads shall be taken from Fig. 6*p*. The concentrated loads are to be considered to occupy a whole panel length of the main roadway to the exclusion of any other live loads (excepting only the electric railway load). For roadways less than thirty (30) feet wide, only one concentrated load shall be used; and for roadways thirty (30) feet or more in width, two concentrated loads shall be adopted. It will be permissible to use a lighter class of loading for the footwalks of a structure than for the main roadway thereof.

In the case of bridges with a portion of the roadway or sidewalks carried on cantilevers projecting beyond the longitudinal girders or arch ribs on both sides of the bridge, the overhanging portion on one side only is to be considered loaded when designing the longitudinal girders or arch ribs, in case the loaded length is one hundred (100) feet or less; but for greater loaded lengths the entire width of the structure is to be considered loaded. Floor-beams of such structures are to be proportioned, first, on the assumption that the centre portion is loaded with either one or both of the overhanging portions empty; and second, on the assumption that the overhanging portions are loaded and the centre portion is empty. Due account is to be taken of the reversing stresses produced by the above loadings.

6. *Impact Loads.*

For steam railway bridges the impact coefficients are to be found by the formula,

$$I = \frac{165}{n\,L + 150},$$

where n is the number of tracks and L is the portion of the span length which must be covered by the moving load in order to produce the maximum stress on the piece under consideration. Fig. 7c shows curves computed from the above formula for one, two, three, and four tracks.

The corresponding formula for electric railway bridges is

$$I = \frac{120}{n\,L + 175},$$

and Fig. 7d gives the corresponding curves.

For highway bridges the formula is

$$I = \frac{100}{n\,L + 200}.$$

In this case n is equal to the total clear width of roadway and footwalks in feet divided by twenty (20). Fig. 7e shows the corresponding curves for $n = 1$, $n = 2$, $n = 3$, and $n = 4$. In case that the value of n be fractional, the impact can be found by interpolation. There is to be no impact for road-roller loading.

7. *Dead Loads.*

The dead load for any member of a structure is to include the weight of all permanent portions of the structure carried thereby. The unit weights of various materials can be taken from paragraph 39 of Chapter LXXVIII. Reinforced concrete will ordinarily be assumed to weigh 150 pounds per cubic foot, including the weight of the steel. Under ordinary climatic conditions, in reinforced concrete bridges no provision need be made for a snow load.

8. *Effects of Arch Shortening.*

The stresses produced in an arch rib by the shortening of the rib are to be figured on the assumption that the rib is loaded with dead load plus half live load plus half impact load over the entire span.

9. *Wind Loads.*

For the unloaded structure, the wind load shall be assumed as forty (40) pounds per square foot of exposed area, which area shall be taken as that seen in elevation, plus fifty (50) per cent of all areas normal to the direction of the wind and shielded by other portions of the structure.

For loaded steam-railway structures, the wind load shall be assumed as thirty (30) pounds per square foot of exposed area, plus a pressure of three hundred (300) pounds per lineal foot on the train applied at a height of eight (8) feet above the base of rail.

For loaded electric-railway structures, the wind loads are to be taken as eight-tenths ($^8/_{10}$) of those specified for loaded steam-railway structures.

The wind load will not be assumed to act in conjunction with either the uniformly distributed or the concentrated highway live loads.

All wind loads are to be treated as moving loads. No percentage for impact is to be added to wind loads.

10. *Traction Loads.*

The total traction load on any portion of a structure is to be taken as a certain percentage of the greatest live load that can be placed on that portion of the said structure. For electric-railway bridges this percentage is to be taken as twenty (20); and for steam-railway bridges it is to be determined by the formula,

$$T = \frac{4000}{140 + L}, \text{ with } T_{max} = 20 \text{ and } T_{min} = 10,$$

where T = percentage,
and L = loaded length in feet.
The values of T may be taken from Fig. 9*e*.

Highway loads are not to be considered to produce any traction load.

No percentage for impact is to be added to traction loads.

11. *Centrifugal Loads.*

The centrifugal load for steam- or electric-railway bridges is to be computed by the formula,

$$C = \frac{W V^2}{15 R},$$

where C is the centrifugal load per lineal foot, W is the equivalent live load per lineal foot, R is the radius of the curve in feet, and V is the greatest probable velocity of the train, to be determined by the formula,

$$V = 60 - 2.5\,D,$$

where D is the degree of curvature. The values of C for curves up to twenty (20) degrees can be readily determined from Fig. 8*b*.

All portions of the structure affected by the centrifugal load are to be figured to carry properly the stresses induced by the said load in addition to all other stresses to which they may be subjected. It is to be assumed as applied five (5) feet above the base of rail, the average centre of gravity of the moving load.

Highway loads are not to be considered as producing centrifugal loads.

No percentage for impact is to be added to centrifugal loads.

12. *Effects of Changes of Temperature.*

The stresses produced in all parts of a structure by a rise in temperature of thirty (30) degrees Fahrenheit and by a fall of fifty (50) degrees Fahrenheit must be properly figured.

WORKING STRESSES

13. *Compressive Strength of Concrete.*

The ultimate compressive strength of concrete, as developed at the age of sixty (60) days in cylinders sixteen (16) inches long and eight (8) inches in diameter, is to be assumed in accordance with the values given in Table 37*q*.

TABLE 37*q*

STRENGTHS OF VARIOUS CONCRETE MIXTURES

(In Pounds per Square Inch)

Aggregate	Proportions			
	1 : 1 : 2	1 : 1½ : 3	1 : 2 : 4	1 : 3 : 5
Granite and trap rock	3,300	2,800	2,200	1,600
Gravel, hard limestone, and hard sandstone	3,000	2,500	2,000	1,400
Soft limestone and sandstone	2,200	1,800	1,500	1,100
Cinders	800	700	600	400

Ordinarily, 1 : 2 : 4 concrete should be used for reinforced concrete structures, and its ultimate strength, found as above, should be taken as 2,000 pounds per square inch; but a lower value should be adopted, if it is possible that soft rock will have to be used for the aggregate.

The use of cinder concrete for first-class construction should be forbidden.

TABLE 37r

INTENSITIES OF WORKING STRESSES FOR CONCRETE

Kind of Stress	Working Stresses	
	In Per Cent of Ultimate Compressive Strength	In Pounds per Sq. In. for Ordinary 1-2-4 Concrete
Compression on concrete in extreme fibres of beams and slabs, in general	30	600
Compression on concrete in extreme fibres of continuous beams at supports	35	700
Axial compression on concrete in columns without hooping containing from one (1) to four (4) per cent of longitudinal steel, due to direct loads only:		
$\frac{l^*}{b} < 12$	20	400
$\frac{l^*}{b} > 12$ and < 20	$32-\frac{l}{b}$	$640-20\frac{l}{b}$
Compression on concrete in columns without hooping containing from one (1) to four (4) per cent of longitudinal steel, due to direct loads, and to bending caused by eccentric loads and deflections of girders:		
$\frac{l^*}{b} < 12$	25	500
$\frac{l^*}{b} > 12$ and < 20	$37-\frac{l}{b}$	$740-20\frac{l}{b}$
Axial compression on the core of hooped columns in which the hooping amounts to at least one (1) per cent of the volume of the enclosed core and having from one (1) to four (4) per cent of longitudinal steel, due to direct loads only:		
$\frac{l^*}{b} < 8$	30	600
$\frac{l^*}{b} > 8$ and < 20	$42-\frac{3l}{2b}$	$840-30\frac{l}{b}$
Compression on the core of hooped columns in which the hooping amounts to at least one (1) per cent of the volume of the enclosed core and having from one (1) to four (4) per cent of longitudinal steel, due to direct loads, and to bending caused by eccentric loads and deflections of girders:		
$\frac{l^*}{b} < 8$	35	700
$\frac{l^*}{b} > 8$ and < 20	$47-\frac{3l}{2b}$	$940-30\frac{l}{b}$
Bearing on concrete	30	600
Shear (diagonal tension) on concrete in beams with horizontal rods only and without web reinforcement	2	40
Shear (diagonal tension) on concrete in beams having at least one-half of the tension reinforcement bent up in the region of maximum shear, but no other web reinforcement	3	60
Shear (diagonal tension) on concrete in beams having web reinforcement of stirrups or inclined bars calculated to carry all of the shearing stress exceeding forty (40) pounds per square inch	6	120
Pure shear on concrete uncombined with diagonal tension	6	120
Bond stress between concrete and plain bars	4	80
Bond stress between concrete and deformed bars	5	100
Bond stress between concrete and threaded bars	10	200
Bond stress between concrete and drawn wire	2	40

*l = unsupported length.
b = least outside dimension of column without hooping, or diameter of core for hooped column.

14. *Intensities of Working-Stresses.*

The intensities of working stresses for concrete shall be as specified in Table 37*r*. They are given in percentages of the ultimate compressive strength at sixty (60) days, and also in pounds per square inch for ordinary 1 : 2 : 4 concrete in which the said ultimate strength is 2,000 pounds per square inch.

The intensities of working-stresses for reinforcing steel shall be taken as follows, in pounds per square inch:

Tension or compression on steel in main reinforcement...	16,000
Tension on steel in web reinforcement..................	12,000

The intensities of working stresses for other materials shall be as specified in paragraphs 48 and 49 of Chapter LXXVIII.

The intensities given in this paragraph shall not be exceeded, except as specified in Paragraph 17.

15. *Coefficients of Elasticity for Concrete and Steel.*

The coefficient of elasticity of concrete shall be assumed to have the values given in Table 37*s* when making stress calculations; and for the purpose of figuring deflections, it shall be taken as 3,750,000 pounds per square inch.

TABLE 37*s*

COEFFICIENTS OF ELASTICITY OF CONCRETE

(For use in Making Stress Calculations)

Ultimate Strength. Pounds per Sq. In.	Coefficient of Elasticity. Pounds per Sq. In.
2,200 or less	2,000,000
2,200 to 2,900	2,500,000
2,900 or more	3,000,000

The coefficient of elasticity of steel shall be taken as 30,000,000 pounds per square inch.

The value of n, or the ratio of the coefficient of elasticity of steel to that of concrete, will, therefore, be taken as 15 for ordinary 1 : 2 : 4 concrete.

16. *Combinations of Stresses.*

For various combinations of the stresses produced by the loads specified in Paragraphs 2 and 3, the intensities of working stresses given in Paragraph 14 may be increased by the percentages given in Table 37*t*.

DESIGN

17. *Fundamental Assumptions for Designing.*

Calculations shall be made with reference to working stresses and safe loads, rather than with reference to ultimate stresses and ultimate loads.

TABLE 37*t*

INCREASES IN INTENSITIES OF WORKING STRESSES FOR VARIOUS COMBINATIONS OF LOADINGS

Case No.	Loading	Percentage Increase in Intensities
1	Any loading singly	0
2	Dead load combined with any other one load	0
3	Any combination of live load, impact load, dead load, arch shortening effect, and centrifugal load	0
4	Case No. 3, combined with either wind load, traction load, or effect of change of temperature	30
5	Case No. 3, combined with either wind load plus traction load, wind load plus effect of change of temperature, or traction load plus change of temperature	40
6	Case No. 3, combined with wind load plus traction load plus effect of change of temperature	50

A section plane before bending remains plane after bending.

The coefficient of elasticity of concrete in compression, within the usual limits of working stresses, is constant. The distribution of compressive stresses in beams, therefore, is rectilinear.

In calculating the moments of resistance of beams, the tensile stresses in the concrete shall be neglected.

Perfect adhesion between the concrete and the reinforcement is assumed. Under compressive stresses, therefore, the two materials are stressed in proportion to their coefficients of elasticity.

Initial stresses in the reinforcement, due to contraction or expansion in the concrete, are neglected.

18. *Effective Lengths.*

The effective length for a slab or a beam shall be taken as the distance from centre to centre of supports, but not to exceed the clear span plus the depth of the beam or slab. Small brackets shall not be considered to reduce the clear span length.

The effective length of a column shall be taken as its maximum unsupported length.

The springing of a fixed-ended arch rib shall be considered to be located within the body of the pier or abutment at a distance equal to about half the depth of the rib from the surface of the said abutment or pier, unless the section of the rib is nearly as great as that of the abutment or pier, in which case the springing is to be taken somewhat lower.

19. *Maximum Unsupported Lengths of Columns and Arch Ribs.*

The maximum unsupported length of a column without hooping shall be twenty (20) times its least width; and that of a hooped column shall be twenty (20) times the diameter of the core.

The maximum unbraced length of an arch rib in a transverse direction shall be twelve (12) times its width.

20. *Stresses in Monolithic Structures.*

The stresses in monolithic structures shall be figured on the assumption that perfect continuity exists at all points (except, of course, at expansion joints). The use of approximate methods of calculation will be permissible; but the character of the stresses at all sections must be correctly determined, and a proper amount of reinforcement must be provided for all sections at which tension can possibly occur.

The stresses in fixed-ended arch-ribs shall be computed by the elastic theory.

21. *T-Beams.*

A slab may be considered as an integral part of a beam which supports it only when there is an effective bond between the said slab and beam, and when there is reinforcement in the slab at right angles to the beam. The width of the flange shall not be taken greater than one-fourth (¼) of the length of the beam, nor shall its overhanging width on each side of the stem be taken greater than four (4) times the thickness of the slab.

When T-beams are continuous over the supports, the sections at the supports are to be figured as rectangular beams.

22. *Shear and Diagonal Tension.*

The following sections shall be considered to be under pure shear only:

1. The section of a slab at the edge of the load-area of a concentrated load.
2. The section of a beam or slab at the edge of a support over which it is continuous.

The following sections shall be considered to be under full diagonal tension:

1. The section of a slab located at a distance equal to half the depth thereof from the edge of the load-area of a concentrated load.
2. A section in the portion of a slab or beam carrying a concentrated load which lies between the load and the support, when the distance of the edge of the load-area from the edge of the support is equal to or greater than the depth of the slab or beam.
3. The section of a beam carrying uniform load located at a distance equal to half the depth thereof from the edge of a support over which it is continuous, and all sections from this point to the centre of the span.
4. The section of a beam or slab at the edge of a support on which it is simply supported, and all sections from this point to the centre of the span.
5. Any section of a wall footing located at a distance equal to or greater than the depth thereof from the face of the wall.
6. Any section of a column footing located at a distance equal to or

greater than half the depth thereof from the edge of the column shaft.

The intensity of the diagonal tensile stress at any section of a beam shall be considered constant from the tensile reinforcement to the neutral axis, and equal in value to the maximum unit shearing stress on the said section. Beyond the neutral axis, the intensity of the diagonal tensile stress shall be assumed to diminish uniformly to zero at the compression face of the beam. Near the points of contraflexure of a continuous beam, it must be assumed that either face may be the tensile face.

At points of maximum diagonal tension, the unit stresses in the tensile reinforcement shall, preferably, be low.

23. *Web Reinforcement.*

Vertical stirrups, when used as web reinforcement, must be spaced not to exceed half the depth of the beam. They must be so well anchored at the tension reinforcement that they can be considered as fully developed at that point; and they must extend to the compression face of the beam, and be fully developed between that point and the neutral axis. Near the points of contraflexure of continuous beams, they must be well anchored to the reinforcement in both faces.

Inclined bars, when used as web reinforcement, must be spaced (horizontally) not to exceed the depth of the beam. They must be continuous with the tension reinforcement, or else must extend a sufficient distance along the tension face to be fully developed at the point of bending. They must extend to the compression face, and be fully developed between that point and the neutral axis. Near the points of contraflexure of continuous beams, they must be fully developed at both faces.

When the arrangement of web reinforcement is not regular, the diagonal tensile stress between two adjacent vertical stirrups or rows of inclined bars shall be assumed to be divided between the two said stirrups or rows of bars in proportion to their respective areas.

24. *Minimum Thickness of Concrete.*

The minimum thickness of footwalk slabs (exclusive of wearing surface) shall be three and one-half (3½) inches, and that of roadway slabs, six (6) inches.

The minimum thickness of other reinforced concrete members shall be nine (9) inches; and, preferably, it shall be twelve (12) inches or more.

25. *Minimum Spacing and Edge Distances of Reinforcing Bars.*

The distance from centre to centre of reinforcing bars shall not be less than twice the diameter of the coarse aggregate plus the diameter of the bars, nor less than three times the diameter of the bars. The spacing of adjacent rows of bars shall not be less than the permissible spacing of bars in each row. The distance from the centre of a reinforcing bar to the edge of the concrete shall not under any conditions be less than the diameter of the coarse aggregate plus half the diameter of the bar, nor less than two diameters of the bar; and for construction under water or below the ground surface, the said distance shall not be less than three (3) inches.

The spacing of the main reinforcing bars of slabs shall never exceed the depth of the slab.

26. *Bends in Reinforcement.*

The minimum radius of bends in reinforcing steel under stress shall be fifteen (15) diameters of the bar, except at hooks, or unless stirrups or other special details be used to care for a portion of the radial stresses.

Whenever the radial forces exerted by bent or curved bars produce tensile stresses in the concrete, stirrups must be provided to care for the said tension.

27. *Splices and Development of Bars.*

The length of embedment required for deformed bars shall be forty (40) diameters of the bar; for plain bars, fifty (50) diameters; and for threaded bars, twenty (20) diameters. A hook formed as shown in Fig. 37*ww* may be considered to develop one-quarter (¼) of the strength of a bar.

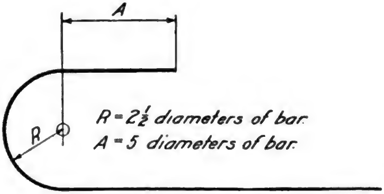


FIG. 37*ww*. Detail of Hook for Reinforcing Bar.

When splices are made by merely lapping the bars, the length of lap must be at least equal to that required for development.

ADDENDUM

Just as the page proof of this chapter was about to be returned to the printer, the author's friend and former assistant, Victor H. Cochrane, Esq., Consulting Engineer, showed him the manuscript of an exceedingly important paper entitled "The Design of Symmetrical Hingeless Concrete Arches," upon which he had been working two years, and which will probably soon be published by one of the technical societies. This memoir will reduce the labor of making arch-ring computations fully seventy-five per cent and will be a most valuable addition to the literature of bridge designing.

CHAPTER XXXVIII

FOUNDATIONS IN GENERAL

The permissible pressures on soils of divers kinds at various depths below water are exceedingly difficult to determine, because the conditions at different locations are so variable. A few experiments on the carrying capacities of foundation soils have been made, but generally the permissible pressure is settled by the engineer's individual judgment. In many cases of pier foundations the minimum area of base which the construction conditions allow will be found great enough to provide a safe and satisfactory bearing, but often the character of the foundation soil is such that enlargement of the bearing area becomes necessary in order to avoid all possibility of serious settlement.

In computing the pressure upon the base of a bridge pier, the total live and dead load (excluding impact) from the superstructure is to be found, to it is to be added the weight in air of the entire mass of pier above the base, and from the sum is to be subtracted the weight of water displaced at the lowest stage of the water. Ordinarily, no deduction should be made for the side friction, as there is no certainty that this acts with anything like its full power until the pier is on the verge of settlement. However, in the case of caissons sunk very deep into soil of small bearing capacity, it may become necessary to rely on side friction to a very great extent, although the less such reliance is invoked the better. In many cases of pile foundations the piles do not reach either bed rock or any very hard material, hence it is often their side friction that affords the main support for the pier and its load. By ignoring side friction on cribs and caissons an error on the side of safety is generally involved, but this is offset more or less by ignoring the effect of impact from the live load. Such impact certainly exists on the spans themselves, but probably never actually to the extent assumed in the specifications. Again, the percentage of impact for a pier would be figured the same as that of a single span having a total length equal to the sum of the lengths of the two spans which the pier supports, hence it would be comparatively small. And, finally, the great mass of the pier absorbs vibration to such an extent that the effect of impact upon the pressure on the base must be considerably smaller than that on the top of the pier.

All things considered, it is probable that for ordinary conditions in foundations of bridge piers the preceding assumptions are as correct as any that can be made; but, when dealing with soils of very small bearing capacity, the bridge engineer must exercise considerable ingenuity to obtain proper supporting power at moderate expense. He will find, of course, that after making an assumption of bearing area for any layout of spans and piers and ascertaining that the intensity of pressure is too great, the more he enlarges the base the greater becomes the total load thereon; and for exceedingly high cribs it often seems that there is very little use in increasing the area, because the total load augments in nearly the same proportion. It is in such cases that one has either to count greatly upon the aid of the side friction or else to adopt some unusual expedient of design for enlarging the base without increasing materially its total load, such, for instance, as making the piers hollow or injecting grouting into the material below and adjacent to the bottom of the caisson.

In 1902 the well known engineer, Dr. E. L. Corthell, made an elaborate investigation of pressures on foundations, and in 1907 he gave the results thereof to the engineering profession in book form under the title of "Allowable Pressures on Deep Foundations." For foundations that gave no sign of settlement he found the following intensities of pressure or loads per square foot:

On fine sand from 4,500 lbs. to 11,600 lbs., with an average for ten (10) cases of 9,000 lbs.

On coarse sand and gravel from 4,800 lbs. to 15,500 lbs., with an average for thirty-three (33) cases of 9,800 lbs.

On mixed sand and clay from 5,000 lbs. to 17,000 lbs., with an average for ten (10) cases of 9,800 lbs.

On alluvium and silt from 3,000 lbs. to 12,400 lbs., with an average for seven (7) cases of 5,800 lbs.

On hard clay from 4,000 lbs. to 16,000 lbs., with an average for sixteen (16) cases of 11,160 lbs.

On hard pan from 6,000 lbs. to 24,000 lbs., with an average for five (5) cases of 17,400 lbs.

For foundations that had settled he found as follows:

On fine sand from 3,600 lbs. to 14,000 lbs., with an average for three (3) cases of 10,400 lbs. In the minimum case the bearing was probably unconstrained quicksand.

On clay from 9,000 lbs. to 11,200 lbs., with an average for five (5) cases of 10,400 lbs.

On silt and alluvium from 3,200 lbs. to 15,200 lbs., with an average for two (2) cases of 9,200 lbs.

On mixed sand and clay from 3,200 lbs. to 14,800 lbs., with an average for three (3). cases of 6,600 lbs.

Of course, every case of foundations has to be judged upon its own merits as it arises, nevertheless it is necessary for a bridge designer to have at hand some intensities of safe loads for ordinary cases. For deep foundations the author offers the following, which are based upon both his own practice and the data collected by Dr. Corthell.

TABLE 38*a*

SAFE LOADS ON DEEP FOUNDATIONS

Materials	Safe Load per Sq. Ft.
	Lbs.
Confined Quicksand	7,000
Other Fine Sand	9,000
Coarse Sand and Gravel	11,000
Mixed Sand and Clay	7,000
Alluvium and Silt	5,000
Hard Clay	12,000
Hard-pan	18,000
Rock	40,000

For shallow foundations the corresponding intensities of safe loading are given in the following table:

TABLE 38*b*

SAFE LOADS ON SHALLOW FOUNDATIONS

Materials	Safe Load per Sq. Ft.
	Lbs.
Fine Sand	6,000
Coarse Sand and Gravel	8,000
Mixed Sand and Clay	5,000
Alluvium and Silt	3,000
Hard Clay	9,000
Hard-pan	14,000
Rock	40,000

In large constructions involving a great number of pier or pedestal bases, such, for instance, as elevated railroads, it is advisable for economic reasons to do considerable experimenting upon earth resistance. The author made several tests of this kind when building the Northwestern Elevated and the Union Loop Elevated railways of Chicago. A description of the apparatus used and the results obtained are given in Mr. S. M. Rowe's discussion of the author's paper on Elevated Railroads, which was published in the 1897 *Transactions* of the American Society of Civil Engineers.

In determining permissible pressures on bases of piers, it must not be forgotten that small areas will generally develop higher unit resistances than large ones. This is probably owing to the side friction, the effect of which upon the intensity of pressure reduces as the area of the base increases. This reduction, however, does not apply to test intensities, because the bearing base of the apparatus is supposed to lie upon the surface of the soil tested, hence there is no side friction developed until the base sinks, and very little even then.

For side friction in sinking cribs and caissons the author usually figures on six hundred (600) pounds per square foot, which is ordinarily enough for penetrating the beds of alluvial rivers. Dr. Corthell finds for sinking cylinders from three hundred (300) pounds through mud to fifteen hundred (1,500) pounds through gravel, and for sinking masonry piers from three hundred (300) pounds through sand to one thousand (1,000) pounds through mixed sand and clay, with an average for twenty-three (23) cases of five hundred and twenty-two (522) pounds. In figuring on sinking cribs and caissons one is naturally interested in the greatest probable frictional resistance, while in estimating on bearing capacity the least probable amount is what is needed to be known.

The three principal methods of pier sinking which are in common use are as follows:

1. The Cofferdam system,
2. The Pneumatic process,
3. The Open-dredging process.

The use of ordinary cofferdams is, or should be, limited to crossings where the bed rock is not more than fifteen (15) feet below the ordinary stage of water, and where there is no great, sudden rise anticipated. This method almost always figures low in the preliminary estimate, but is generally found to run much higher when the total cost of the finished structure is computed. The author makes a practice of discouraging his contractors from attempting to employ this method, except where the conditions are unusually favorable; and thus far his experience proves that the advice is sound. Cofferdams are liable to give trouble in several ways: first by leakage, second by flooding, and third by collapsing. If a contractor gets through a large piece of coffer-dam foundation work without accident or trouble of some kind, he is in great luck. For bare bed-rock movable cofferdams may be employed; but they are troublesome to construct, and are sometimes very difficult to remove on account of a deposit of sand or silt taking place while the piers are being built.

The pneumatic process for sinking caissons is in most cases the best one to employ, the only objection to it being the excessive cost of installing the plant, even if one has a complete outfit at his disposal. Its great advantages are that it enables the contractor to overcome, in the cheapest and most expeditious manner possible, all obstacles that may be encountered in sinking; and that it ensures the obtaining of a satisfactory foundation for the caissons. It can be used for depths as great as one hundred and ten (110) feet or even more, although there is danger to the workmen when the depth exceeds ninety (90) feet. A large proportion of the bridge piers which the author has built have been sunk by the pneumatic process; and he has no hesitation in recommending it as the most satisfactory method for most of the cases of deep foundations which occur in a consulting bridge engineer's practice.

The open dredging process is suitable for very deep foundations, or for putting down caissons that are to rest on the sand, or for bed-rock foundations that are not liable to great scour. For large piers this process is much cheaper than the pneumatic on account of both the smaller cost of plant and the more rapid progress in sinking. In case, however that obstacles be encountered, such as trees or large boulders, the expense for sinking is liable to run high, as these obstacles have to be removed by a diver or divers, which always involves great expense. The open dredging process is liable to abuse by the builders of cheap highway bridges, who, in order to save a little in first cost, use it to sink cylinder piers of small diameter moderate distances to bed-rock, which may in these places be laid bare or nearly so by excessive scour. With this process it is generally not practicable to anchor the cylinders firmly to the bed-rock, but with the pneumatic process it is.

Each of these three methods of sinking is treated in a separate chapter devoted exclusively to the subject.

There is still another type of foundation besides those described, viz., that which involves the use of piles. This may be divided into two classes: first, that in which the piles support a timber grillage to sustain the shaft; and, second, that in which the heads of the piles are encased in a mass of concrete, upon which the shaft rests. The first method is cheap but objectionable, for the entire support of the pier and its burden must be by means of the piles alone; and as these cannot be sawed to perfect elevation, some of them are sure to carry more weight than their proper share. Again, there is a division plane that is simply an invitation to shear the shaft from the bearing between the neat work or base and the grillage, because the concrete does not adhere at all closely to the wood; and there are other planes of division also between the adjacent layers of timber and between the lower layer and the pile heads, in which, besides friction, the only resistance to shear is that afforded by the drift bolts. As there may not be any of the latter driven into the pile heads, and as, if an attempt be made to put them in, some of them may miss their mark, it is evident that this type of foundation is anything but satisfactory in case the piers are subject to jams of ice or logs. Finally, any scour that there may be in the future is almost certain to reduce the bearing power of the piles. It is only inexperienced engineers who adopt or advocate this type of construction in bridges of any importance; because the other type costs but little more and is far superior in every respect.

In the type in which the piles are encased, the pile heads are so held together by the mass of concrete that all the piles have to act in concert, even for eccentric loading, and there is no possibility of the existence of any initial improper distribution of load. The continuity between the shaft, the base, and the supporting piles forms an integral construction which is as truly first-class as such a cheap type of work can be made. In piers of this kind the cutting edge of the crib or box which contains the concrete should go down several feet further than the greatest possible scour can reach; because, if the scour ever does go below the concrete base, the piles forever afterward will have to carry practically the entire load. Even if the scoured space be filled again by deposit, the material thus washed under the base cannot take up any load worth mentioning. The piles for foundations of this kind should either be driven to refusal by the ordinary means or be sunk by water jets to as great a depth as practicable. The methods of driving foundation piles are treated fully in Chapter XLII.

The question as to when pile foundations for bridge piers should and should not be used is not always an easy one to settle; but, in general, it may be stated that they should never be adopted where there is any real danger of their being scoured out. Of course, in light highway bridges, where money for the construction is very limited, it may be wholly impracticable to adopt expensive pneumatic or open-dredged piers, in which case pile foundations may have to be used, even if some risk be run; but in railroad bridges no chance of disaster which can be avoided by the expenditure of any obtainable amount of money should ever be taken.

To determine with certainty whether a river is likely to scour much around the piers of a proposed bridge is a problem requiring experience and engineering judgment of the highest order. One should study thoroughly the river both above and below the site and take soundings over a long stretch of its length in order to discover whether the current has excavated any deep holes. The fact that at a low stage of water none are found is no conclusive proof that none ever exist, because the holes that are excavated during flood are often filled very quickly by sand deposit as the water subsides. One must study the character of the river and that of its bed in order to determine the liability to scour; and he should remember that the contracting of the cross-section by the piers will accentuate any tendency there may be to deepen the channel during high water. This question is very fully discussed in Chapter XLIX.

To make a choice for any bridge between open-dredged and pile piers it is necessary to determine by judgment the least absolutely safe depth for the base of the caisson and that for the base of the pile box, also the requisite penetration of the piles below the said box, then proportion both piers and compute their costs, the cheaper design being adopted. The base of the open-dredged pier should be materially lower than that of the pile one—say from ten to twenty feet, according to the character of the soil and other physical conditions; but the area of the former may be less than that of the latter, which has to contain a number of piles spaced not much less than three (3) feet centres, the said number being determined by the surmised, or possibly the tested, safe load per pile. It is often difficult to settle in advance of the driving what should be the length of foundation piles, and in cases of extreme doubt a few of the longest obtainable should first be driven, then any unnecessary projections should be cut off, after which the remainder of the order can be placed for the proper gross length, care being taken to keep on the side of safety, because a pile that is too short usually cannot be relied upon to have much bearing value.

If the base of a pier having a pile foundation lies above the elevation of extreme low water, the piles should be either of reinforced concrete or creosoted timber, preferably the former. If untreated timber be used, it is possible that air may penetrate to the portions of the piles between the elevations of pier base and extreme low water and cause them to rot, thus letting the pier settle. It is also possible that the creosoting of the piles will merely put off the evil day; but it will be to an exceedingly distant date—so distant, probably, as to extend into some future century.

In pier foundations in salt or brackish waters where the *teredo navalis* or other sea worms exist or are ever likely to exist, no piles or other timber should be so placed as ever to be accessible to their attacks. If there is to be in the future any deepening of the channel, all timber work necessary to the safety of the construction should be sunk well below the anticipated depth of such possible excavation. The author has encountered three cases of this kind in bridges that he has built over False Creek for the City of Vancouver, British Columbia. In these, certain pneumatic caissons had to be sunk at considerable expense a number of feet into the hardest kind of cemented gravel and boulders, in order to bring the top of the timber deck of the working chamber below the proposed bottom for a thirty (30) foot channel. The periphery of the cribs will probably in time be destroyed by the sea-worms; but the interior will remain intact, as the bracing timbers of the shells were removed during the filling just before the concrete reached them.

Occasionally a caisson that is being sunk through sand or gravel reaches such a hard material that it is impracticable to sink into it, and there is not enough penetration to afford the requisite lateral stability. In such a case it becomes necessary to place around the base a mass of both large and small stones so as to resist properly all tendency to overturning. This was done on one of the piers of the author's Fraser River bridge at New Westminster, British Columbia.

Occasion might sometimes arise for building a bridge on a thin crust of comparatively hard material overlying a far softer material of great depth—such conditions, in fact, as exist in the City of Mexico. The proper way to design the piers under these circumstances would be to make them as light as possible, carrying the shaft down at the usual batter to near the surface of the ground and spreading it suddenly to large dimensions by means of a reinforced concrete slab or by resting it on a large steel grillage encased in concrete so as to protect the metal against rusting. By this means the total load on the pier base could be spread over such a large area that the intensity of pressure on the soil would be reduced to a safe amount. This type of foundation was originated by the author for important buildings in the City of Mexico early in 1900, and was described by him in both English and Spanish in the leading newspapers of that City. In 1905 this description was reproduced by Mr. Harrington in his collection of the author's "Principal Professional Papers," to which the reader is referred.

In case a pneumatic caisson rest on a soil of insufficient bearing capacity, it is feasible to enlarge the base somewhat and thus reduce the intensity of pressure by excavating one at a time and filling with concrete (either plain or, preferably, reinforced) small trenches in the bottom and extending them on each side as far beyond the cutting edges as practicable. In this manner a platform of reinforced concrete two (2) feet or more in thickness and projecting about that distance beyond the sides of the caisson may be made to occupy the entire base. This expedient, however, is not altogether a satisfactory one, because the soft material will tend to prevent an effective junction of the contiguous beams, and much dirt is likely to become mixed with the concrete, thus reducing its strength.

In founding a pneumatic caisson on an area of varying resisting capacity it often becomes necessary to reinforce the weak places. This may be done by putting in bags of concrete, mixed rather dry so that the mass will not spread unduly, the necessary moisture for the concrete being absorbed from the surrounding soil. Sometimes a caisson lands on a sloping surface of rock that is much higher at one end or side than at the other. Here it is necessary either to cut deeply into the rock at the high places or to excavate below the cutting edge to the rock at the low places and build up by bags of concrete. Such work often requires great care and skill.

The method of increasing the bearing power of soil by injecting grouting into it under pressure has been tried more or less successfully on several occasions. Where the material is gravel the scheme works well and the grout penetrates outward as much as ten (10) feet; but where the material is fine sand the penetration is small, sometimes not to exceed a few inches. Unfortunately, the method is much more likely to be needed for a sand foundation than for one of gravel, because the latter material at great depths where it is beyond all possible reach of scour may be loaded to almost any practicable amount without causing settlement. It is in the soft materials of alluvial streams where enlargement of bearing areas becomes necessary, and in these the success of the method is problematical.

A modification of this method was tried in India by H. F. White, Esq., Mem. Inst. C. E., who excavated several foundation pits under water, deposited in each a layer of broken stone from a foot and a half to seven feet deep, and injected grouting into it by means of a two-inch pipe. After allowing the concrete thus formed to set, he pumped out the water and made an examination of the work, finding that the mass was filled fairly well. He discovered that in order to make the mortar flow satisfactorily, it was necessary either to mix lime with the cement or to use lime alone. As lime, in the author's opinion, is not a fit material for bridge construction, he does not consider Mr. White's experiment a success. Moreover, it would have been far easier and far less expensive to deposit at the outset rich broken-stone concrete by trémie or collapsible box. This is the author's invariable method of filling caissons and cribs where the water is not removed. On p. 373 *et seq.* of Jacoby and Davis' excellent book entitled, "Foundations of Bridges and Buildings" will be found some information of value concerning the "Grouting Process," and the reader is advised to peruse it.

There is another means of sinking piers that should not go unmentioned, viz., the "Freezing Process." It was originated some thirty years ago in Germany by Dr. F. H. Poetsch, but has never been utilized for bridge piers, although there are locations where it might be found applicable. It consists in sinking a number of pipes about five (5) inches in diameter and three (3) or four (4) feet apart over an area somewhat greater than that of the foundation required, driving them as low as the final elevation of the base, removing the contained material, placing inside them other pipes from one (1) to one and a half (1½) inches in diameter, and forcing down the small pipes and slowly up the large ones a freezing mixture. This continued action freezes the ground solid after the expiration of about a month. By judicious location of the pipes around the outside and by insulating the inner ones nearly to the bottom, a shell of solidified earth is formed and the water is shut out by the congealed base, thus permitting the interior material of the said shell to be easily excavated in the dry and the pier base to be built in the well thus formed. The process is exceedingly slow and is likely to be expensive; hence it should be adopted only where the pneumatic and the open-dredging processes are not applicable. The freezing process was patented in the United States over twenty years ago, hence the original patents have expired and the method is available for the use of anyone, unless, perchance, later claims for improvements have been granted.

In concluding this chapter it is appropriate to mention the extreme importance of having invariably for bridge piers foundations that are safe beyond the peradventure of a doubt; for if the foundation of any pier fails, the bridge is almost certain to be destroyed, and with it possibly many human lives. Unscrupulous bridge contractors are prone to slight the foundation work in preference to the superstructure, because the former is out of sight, while the latter is always in evidence; but if they have to neglect one portion or the other, it would be much better to choose the superstructure, as that can generally be remedied, while defects in the foundations are either irremediable or can be rectified only at excessively great expense.

CHAPTER XXXIX

COFFERDAMS

In constructing foundations for bridges, water is usually encountered; so that the engineer must make provision for excluding it from the space to be occupied by the construction, or else must adopt a type of foundation that can be placed under water. One of the earlier and more common means of excluding water from substructure work was the building of a wall of earth, stones, or timbers around the space to be occupied by the foundation and then pumping the water out. This left the bed of the stream exposed; so that workmen could clean off the bed-rock. or make the necessary excavation to reach a solid stratum.

With the cumulative experience of bridge builders, many variants of this original type of cofferdam have been developed. Indeed, development has proceeded so far in some lines that the resulting type is no longer classified as a cofferdam, but as an open crib or a pneumatic caisson. The distinction that will be made in this chapter is that of permanency of position; the cofferdam remains until removal as first placed in position, while the open crib and the pneumatic caisson sink into final position as excavation proceeds and eventually become a part of the permanent structure.

Cofferdams can be used advantageously for shallow crossings where the bed-rock is near the surface, and where no great, sudden rise in water level is anticipated. There are instances of cofferdams being successfully used for depths of thirty-five feet, but the author recommends that their adoption should generally be limited to fifteen feet, unless steel sheet piling be employed.

The various types of cofferdams are as follows:

1. Earthen dams for shallow water and sluggish current. This type requires a relatively large amount of material to secure stability and imperviousness. It invariably occupies a large space for the amount of work room afforded, and should be considered only when there is plenty of area available and when material is close at hand which can be readily obtained and cheaply placed.

2. Rock dams with earth filling and covering to prevent leakage. These have the advantage over strictly earthen dams, because the same stability can be secured with a smaller cross section and a consequent reduction in the amount of material used. This, in turn, means more work room for the total space occupied and generally involves a reduction in cost.

3. Bags filled with clay and sand and deposited in an orderly way

on top of each other so as to form a wall around the space to be occupied by the foundation. This method has been employed with success. Mention should be made of the possibility of removing these bags of earth and using them over again at another foundation.

4. Log cribs, made directly from felled timber and surrounded on the outside by a ramp of earth. These are adapted for some locations where timber is plentiful and suitable earth is convenient to the bridge-site.

5. Dams made by driving a row of sheet piling around the space to be occupied. This type is suitable for locations where bed-rock is overlaid with a softer material that can be penetrated by the pile without too hard driving. The piles may be of the wooden tongue-and-grove type or of the interlocking steel type. With the latter it is possible to go to greater depths. There are some cases on record in which sixty-foot depths were attained.

6. Dams made by driving a double row of sheet piling and filling the intervening space with a suitable earth puddle. This dam is better adapted for great depths where the lateral pressure is too much for the single row of piles.

7. Reinforced concrete crib, having double walls to secure buoyancy, made on shore, and then floated into place and weighted until it sinks into a previously prepared bed of clay which seals the bottom.

8. Dams made by freezing a wall of water *in situ*. In addition to its low cost, the ice dam is comparatively free from the danger of a blow-through; and the leakage is reduced to a minimum.

Should the engineer decide to adopt the cofferdam system in preference to the pneumatic or the open-dredging process, there yet remains the question of what type of cofferdam it is best to select. In making a choice one must take into consideration the nature of bed-rock or other foundation; the overlying deposits of silt, clay, or gravel; the availability of the various materials and their costs; the probability of floods and ensuing damage to structure; the obstruction of the stream and interference with navigation; and the possible consequential damages to property owners near the works.

Dams of the first four types involve the use of earth to shut out water. Unless a puddle wall is provided, earthen dams must be made of sufficient extra width to withstand saturation and prevent seepage. The slope of the faces should be three horizontal to one vertical in order to secure stability for the saturated material; and the top of the earth dam should be two or three feet higher than the working stage of water. The best puddle material is a lightish loam mixed with fine gravel or coarse sand. Clay should have sand mixed with it; for otherwise it will absorb too much water and then shrink and crack when it dries out. Again, a leak in clay does not mend itself as it does in the other materials. Top soil, because of the roots and vegetable matter contained, is not suitable for shutting out water. Puddling should be placed in layers about a foot thick and worked over with sufficient water to form a plastic mass that can be packed and crowded into all corners, cracks, and openings.

In constructing a dam of type No. 3, care must be taken not to fill the bags too full or they will not pack well and will not make a watertight dam. The thickness of the wall must be adjusted to correspond with the depth of water. As a guide to one's judgment in deciding this point, the wall may be considered as being composed of a material weighing forty pounds per cubic foot and tested at any assumed section both for stability against overturning and for sliding. The coefficient of friction may be taken as 0.5.

Type No. 4 permits of a vertical inside face being obtained without expensive construction. This is desirable, as it reduces the total space occupied by the cofferdam and involves less water to pump. The crib, which can be constructed on shore and floated into place, should be made somewhat larger than the foundation so that sufficient clearance will be allowed for possible distortion and displacement during the process of locating and for working room for the suction hose of the pump and for building the base of the construction.. The placing of the earth ramp around the crib after it has been lowered into final position should proceed fairly uniformly on all four sides so that there will be a balancing of pressures. This ramp of earth must be of sufficient thickness to prevent any seepage from starting; for if percolation should commence, it would carry out the fine particles of soil, thereby causing a rapidly enlarging hole and a corresponding flow of water. The minimum width at the top of the ramp should be three feet. In estimating the amount of earth required, it should be remembered that the angle of repose for saturated clay mixed with sand is only about half of the angle that exists with the same material dry. Interior bracing will be required to prevent distortion and possible collapsing of the crib. This bracing is usually made of horizontal struts arranged in tiers and held in place by wedges driven between their ends and vertical timbers that distribute the pressure of the individual logs. As the base of the construction is placed, the lower struts can be removed by knocking out the wedges from time to time as the work progresses.

Cofferdams of type No. 5 have to be constructed in place. To make it feasible to drive sheet piling, a suitable frame work has to be prepared at the site of the pier. This frame work may consist of horizontal wales bolted to round piles driven into the material overlying the foundation, or it may consist of a skeleton crib floated into place and weighted down into the river bed. Where the overlying material is insufficient or too soft to give stability to the guide piles, the skeleton crib is to be used. Until the advent of the tongued and grooved sheet pile of the Wakefield type, separate planks were driven as close as possible to each other, making an enclosure around the previously prepared guide frame. To reduce leakage, additional planks were driven outside to cover the cracks, or a double row of planks was put down so as to overlap the said cracks. With the best of care in driving, the planks do not close up tight enough and the operation usually ends by resorting to a canvas covering or an embankment of earth. The introduction of the Wakefield sheet pile gave more satisfactory results as far as the vertical cracks were concerned; but, owing to irregularities in bed-rock, considerable leakage sometimes occurred at the bottom. This can be reduced by sharpening the piles on the edge and doing a little extra driving so as to broom the bottoms and make a better fit with the rock.

Unless there is an overlying plastic material to give proper penetration to the pile, it is more satisfactory and cheaper to use an open crib or box, as described further on.

The double row of sheet piling with puddle between has also to be constructed in place. Unless there is an overlying material to afford sufficient penetration to fix the bottom ends of the sheet piles, much difficulty is encountered in bracing the outer row of piling to the inner. This condition is best met with the crib construction referred to in the previous paragraph and described later on. However, should there be a suitable material for holding the bottom ends of the piles in position, the engineer should proceed to build the guide frames for both inner and outer rows of piling. The piles should be sharpened to a bevel on the bottom edge so that there will be a tendency to close up on each other during the driving. The amount of space to be allowed between rows of piling will vary inversely with the degree of stability afforded by the guide frames and with the amount of workroom needed for derricks, pile drivers, and other equipment. A comparatively thin wall of puddle when properly made of suitable materials will provide against leakage. Puddle walls only four feet thick have been successfully used against heads of thirty-five feet; and for the greatest limiting head recommended at the beginning of this chapter a two-foot puddle wall, if properly supported, will prevent the percolation of water. The practical difficulty is to prevent subsidence and rupture of the puddling. A through bolt, or timber brace, or any smooth surface invites the passage of water, which, when once established, causes a larger opening and an increasing flow.

For stopping a leak, the method known as "stock ramming" is effective. It consists in the boring of a three- or four-inch hole through the sheet piles in the vicinity of the leak and pushing into the hole a previously prepared cylinder of clay which is to be rammed in tight with a specially constructed rammer. This rammer is made of a pole fitting the hole easily and provided with a leather flap at the foot. A hole bored longitudinally in the rammer, leading from the leather flap to a cross hole some twenty or thirty inches farther back, provides access for the air when withdrawing the rammer, thus overcoming the suction.

Many of the objections to cofferdams as a means for constructing foundations have been overcome within the last fifteen years by the introduction of steel sheet piling. They will stand hard driving; and being harder than many bed-rock strata, they can be driven into such bed-rock a sufficient distance to fix firmly the bottom ends and thus stop the underflow. Being formed so as to interlock with each other, the vertical joints can readily be made water-tight by inserting wooden strips, sawdust, or paper pulp, or by pouring in a coarse grout. As they are of superior strength to the wooden sheet piles, the need for bracing is much reduced. The interlocking feature also renders unnecessary the horizontal guides; and they can be driven to conform readily to the various shapes of different footings. They can be pulled out and used over again many times on the same and other jobs.

The application of reinforced concrete to cofferdam construction has been proposed. As early as 1906, it was considered as a means for closing the outer basin of Port La Rochelle. A design and estimate were made and the cost was compared with that for a timber cofferdam. The comparison showed that the latter was the more expensive. It was proposed to construct the reinforced concrete dam with double walls so that it could be floated into place and weighted down until it sank into final position. Another novel type of cofferdam to be constructed by the freezing process was proposed for this work. The plans called for the freezing of the water *in situ* and maintaining the ice until the completion of the job. A comparison of estimated costs indicated that this method was cheaper than either the timber puddled cofferdam or the reinforced concrete cofferdam. In the freezing process it is necessary to enclose the wall of water in a sheeting of non-conducting material. Which of the three competitive types of cofferdam was chosen the author has been unable, after much search, to determine; but the comparison is just as interesting notwithstanding.

There is another type of construction which serves the purpose of a cofferdam, though strictly speaking it should not be included in that class. This is the crib, or open box, used in connection with pile foundations. The author has employed this extensively even in some important structures when conditions were favorable.

The crib type does not depend on a packing of earth to make it watertight. Being constructed of sawed timber drift-bolted together, the cracks and joints are small and can readily be caulked with oakum from the outside before the crib is launched from the shore. In shallow water this is sufficient, but further imperviousness can be secured against the higher heads of water by sheeting the outside of the crib with tongued-and-grooved planks nailed on vertically. Heavy canvas is sometimes used for the outer covering, a wide flap being left at the bottom so as to fit the irregularities of the bed of the stream and check the inflow of water. This flap is weighted down by attaching a heavy chain to the edge of the canvas before the crib is launched. Care must be taken to have the flap extended horizontally outward from the crib when it lands upon its bed. This crib will require removable internal bracing. For pile foundations this type of cofferdam is very suitable. After the necessary excavating and cleaning out have been done, the crib can be flooded, thereby reducing the lateral pressures, and then the piles can be driven through water, cutting out the expense of pumping for that period. After the piles are driven, short struts should be placed between the pile heads and the walls of the crib as the water is again pumped out. This permits of the removal of the longer struts originally placed and gives more workroom for final cleaning out and concreting. It will be found that the driving of the piles displaces the soil or gravel upward so that it will be necessary to take out some more material to reach the desired elevation for the bottom of the concrete base. This is especially true if a water jet be used in connection with the driving of the piles.

If the crib should land on an impervious layer, so that the bottom edge penetrates the soil somewhat and shuts off the underflow, it can be pumped out and the concrete can then be placed in the dry. However, should the crib land on a pervious stratum, the water will flow in at the bottom, so that continuous hard pumping would be necessary to keep the water level down. The placing of concrete under such unfaorable conditions is to be avoided, for the current created by the pumping will cause a serious loss of cement during the entire process of deposition. Seldom can the pumping proceed continuously; hence the flooding of the fresh concrete due to any shutting down of the pump means stopping the work to allow the concrete to harden, or to draw off the water again and with it a portion of the fresh cement. To overcome this difficulty, the water may be allowed to stand in the crib at its normal level and the concrete may be deposited through it by means of a trémie or a trap box. Owing to the pile heads projecting upward into the crib, there is little room to work the latter properly; hence the trémie is preferable. Unless care be taken, the trémie will frequently lose its load and have to be recharged by dropping into it fresh concrete through the water, thus causing a certain amount of segregation of the sand, cement, and rock. If a trap box is employed, it will frequently catch on a pile head and be upset, thereby permitting the load to fall through the water and inducing segregation. The result of either difficulty is the loss of homogeneity in the concrete and a resulting decrease in the strength of the base. These troubles and defects can be obviated by building the crib somewhat larger than the footing, so that a smaller form can be placed inside of the crib, thus leaving a nine to twelve inch water channel on the four sides between the inner form and the crib. A sump should be dug in one corner of the crib for the strainer of the suction hose. If this sump is deep enough and if the capacity of the pump is sufficient, the water level can be kept down to or a little below the top surface of the bed of gravel, so that the concrete can be placed in the dry within the inner form. As the lower layers of concrete set up sufficiently to resist wash, the water level can be allowed to rise gradually, thereby decreasing the duty of the pump. This method permits of each batch being deposited in full view of the supervising engineer, and no uncertainty can then arise as to the quality of the concrete.

After the cofferdam is completed, it becomes necessary to pump out the water so that the foundation work may proceed. Where large volumes of water are to be raised against low heads, the centrifugal pump is the best kind, especially as it can operate with considerable sand and silt in the water. This type of pump must be kept running above a certain critical speed or it will lose its priming. For a limited amount of water, the pulsometer is best adapted. The height to which it can lift water is practically unlimited. It can be hung inside the cofferdam or crib and does not require much room.

The difficulties usually encountered in the cofferdam method of placing a foundation are many. There is the constant leakage to be removed; for it generally requires too large an expenditure of time and money to make cofferdams absolutely water-tight. There is the constant danger of flooding with its damage to a partially completed substructure, as well as a halt in operations. There is also the possibility of collapse with consequent damages and often with the loss of life. Even steel sheet pile cofferdams have failed in this manner. The usual unevenness of bed-rock and crevices in the same make it difficult to shut out the water, if there is much head to work against. If bed-rock is absent and the substructure has to rest on a gravel formation, there will be a large influx of water from the bottom, requiring a large pumping capacity and constant vigilance to keep the water down so that operations can be carried on uninterruptedly. Should springs be encountered in the bed, it would be best to let the crib fill and then deposit some concrete under water in order to seal the bottom. This concrete should be allowed to set several days before pumping out the crib. In case of small springs it is practical to pipe their flow away to a sump hole and then deposit concrete over the pipe leaving it embedded therein. Under no circumstances should the flow be allowed to go unconfined and spread through the mass of concrete, thereby washing out cement and leaving bare surfaces and weakened bearings in the interior of the base.

As conditions are seldom alike in any two cases, it is impossible to say without a special study of all of them at a particular crossing what the cost of the cofferdam method would likely be. As stated in the preceding chapter, this method nearly always figures low in the preliminary estimate, but is generally found to run much higher when the total cost of the finished structure is computed. While it is possible to calculate with reasonable precision the original amount of material and labor entering into the construction of a cofferdam, one cannot foresee how many times such material and labor may have to be replaced; hence in any cofferdam estimate it is well to allow a large percentage for contingencies.

The guiding principle in selecting a type of cofferdam should be to make the cost of construction, maintenance, and pumping a minimum, provided, of course, that the desired excellence of the permanent construction be attained.

While the author has used in his practice all the preceding types of cofferdams very extensively for small bridges and for the approaches to large ones, or, more strictly speaking, has countenanced his contractors in so doing, he has nearly always avoided employing any of them for the foundations of large structures. A notable exception was his Maumee River Bridge near Toledo, Ohio, where all the piers were put in by means of movable cofferdams, the conditions there being specially favorable for that type, for there was almost no sand deposit, the rock was nearly bare and always at a depth small enough to render inconsiderable the risk of serious trouble, the piers (with the exception of the pivot-pier) were alike, and the distance to bed-rock was practically uniform.

If there be any choice between the employment of open-dredged boxes or cribs, either with or without piles, as described in the preceding chapter, and any one of the various types of cofferdam, the author will adopt the former; because it is generally much more reliable and in the end less expensive. There are conditions, however, where it ought not to be used, for instance, when the bottom is irregular in either form or hardness—or both, or where trouble may be anticipated in sinking the box to a satisfactory depth without pumping out the water. There are cases, though, in which the box can be sunk far enough into a water-tight material to permit of its being pumped out and the excavation continued in the dry without further sinking. This was done on several of the author's British Columbia bridges, including some over the Thompson River—a stream that is both difficult and expensive to bridge because of the varying and irregular characteristics of the bed, the prevalence of boulders. and the occasional extreme hardness of the clay encountered.

Anyone desirous of studying further the subject of cofferdams, or, for that matter, any other branch of bridge pier foundations, is advised to read the third edition of C. E. Fowler's excellent work entitled a "Practical Treatise on Sub-Aqueous Foundations." It contains a vast fund of valuable information upon substructure work in general.

CHAPTER XL

OPEN-DREDGING PROCESS

This chapter will be limited to the consideration of open-dredging as applied to deep foundations, and will not cover cofferdam work, shallow foundations, or open-dredging for pile foundations.

The method is suitable for two conditions only, viz.:

First. Where the foundation is too deep to be reached by the pneumatic process, and

Second. Where the foundation at the depth desired to be attained is sand, gravel, or other hard material that is protected thoroughly against scour.

The advantages of the open-dredging process as compared with the pneumatic process for deep foundations are as follows:

First. Greater cheapness because of

A. No expensive pneumatic machinery to purchase, ship to and fro, and keep in repair.

B. No expensive sand-hogs to employ.

C. No danger to workmen from compressed air.

D. Greater daily progress in sinking.

Second. Greater depth obtainable.

Third. Greater effective weight for sinking; because the upward pressure of the air in a pneumatic caisson, which is about equal to the weight of the water displaced by it, acts as a direct decrease of the effective load for overcoming friction. This remark applies specially to piers of small area.

Its disadvantages are as follows:

First. Uncertainty about the removal of obstacles.

Second. Possible expense of having to employ a diver.

Third. Impracticability of sinking into bed-rock.

While in most cases of deep foundations where there is any choice between the open-dredging process and the pneumatic process, the former is likely to prove the cheaper, it is wiser not to adopt it unless the final bearing be sand or gravel; because it is better to pay a little more for the certainty of sinking which the pneumatic process ensures. Again, it would not be good engineering to adopt the open-dredging process for a rock foundation that is easily reachable by the pneumatic process, because the cutting edges of the caissons would be almost certain to take an uneven bearing on the rock, and this could not be rectified except by the use of divers at a great expenditure of time and money. Moreover, if the bed-rock is higher on one side of the caisson than on the other, it is liable to cause a tipping of the pier, which is very difficult to overcome in such cases without the aid of compressed air. The open-dredging process is specially suitable for caissons of bridges to be built in foreign countries, where the cost of shipping the pneumatic outfit to and fro would be large, and where the repairs to it would be slow, expensive, and difficult to make, provided, of course, that the foundation conditions are satisfactory.

It is difficult to give the comparative costs per cubic yard for sinking caissons by the open-dredging and the pneumatic processes; for so much depends upon the magnitude of the construction, the location of the bridge, the proportion of actual excavation to total mass of cribs and caissons, the nature of the materials penetrated, the amount and character of obstructions, and various other conditions. Under the most favorable circumstances for the open-dredging process the actual cost of sinking per cubic yard of mass of cribs and caissons is about one-half (½) of that for the pneumatic process; for fairly good conditions it is from two thirds (⅔) to three-quarters (¾); for unfavorable conditions it is about the same; and for unusually bad conditions (such, for instance, as many large logs encountered) it is sometimes more expensive.

Open-dredging caissons can often be sunk eight (8) feet per day, which is about as fast as the cribs can be built up, while one-half of that amount is a good record for pneumatic work. The actual sinking by open-dredging often costs only two (2) or three (3) dollars per cubic yard of mass, while five (5) dollars per cubic yard is a fair allowance for the pneumatic process.

The open-dredging process for deep foundations has been in use only about thirty years, the oldest examples of it being the Poughkeepsie Bridge over the Hudson River, where a depth of one hundred and thirty-four (134) feet below high water was reached, the Morgan City Bridge over the Atchafalaya River, where eight (8) foot cylinders were sunk to a depth of one hundred and twenty (120) feet below high water, and the Hawksbury River Bridge in Australia, where the remarkable depth of one hundred and sixty (160) feet was attained.

Probably the greatest depth ever reached was on the bridge over the Ganges River at Sara, India, the cutting-edge of one of the piers for this structure landing one hundred and sixty (160) feet below lowest water, or one hundred and ninety (190) feet below high flood level.

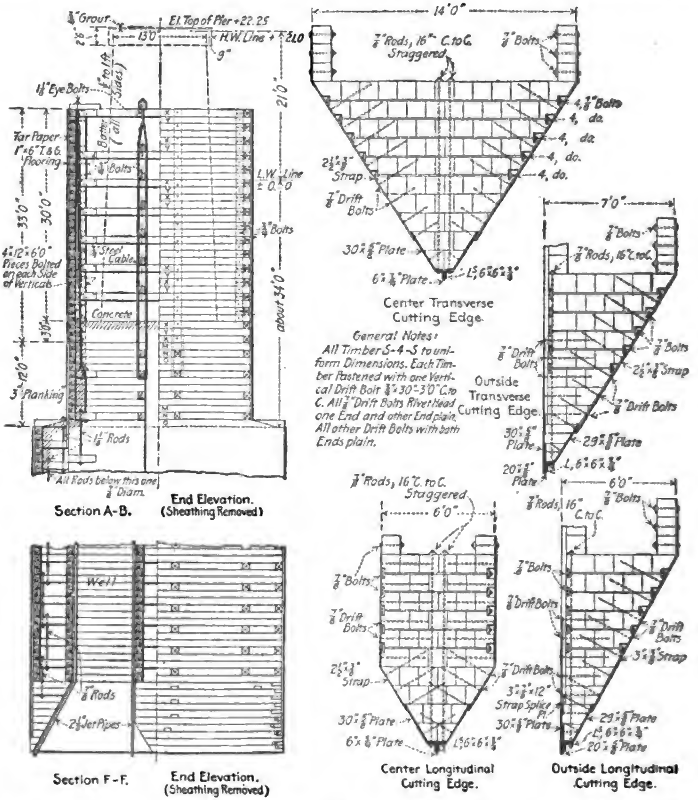
The engineer in this country who has employed the open-dredging process for deep foundations the most often and most successfully is probably the author. He first used it in the early nineties on the Missouri River at Sioux City and at East Omaha. In the former case the depth below high water attained was one hundred and ten (110) feet, and in the latter one hundred and forty-two (142) feet. About 1900 he employed the process for bridges over the following important rivers on the line of the

Vera Cruz and Pacific Railway in the Republic of Mexico: Papaloapam, Tesechoacan, Colorado, and Trinidad. In these cases the foundations were sand and gravel, with no bed-rock discoverable by the boring outfit; and the depths below extreme low water averaged about fifty (50) feet, being sufficient to ensure the piers against all possible scour.

In 1902 and 1903 the author engineered the building of a bridge over the Fraser River at New Westminster, British Columbia, in which some of the piers were sunk by open-dredging to a depth of one hundred and twenty-seven (127) feet below extreme low water, or more than one hundred and forty (140) feet below high water.

In the Oregon and Washington Railway and Navigation Company's vertical lift bridge over the Willamette River at Portland, Oregon, designed and engineered by the author's firm under the direct supervision of Mr. Harrington, the two main piers shown in Figs. 40*a* and 40*b* were sunk by the open-dredging process under great difficulties. The depths to which their bases had to go, viz., 132 and 145 feet below low water, rendered the open-dredging process obligatory. In plan, each caisson was 36 feet × 72 feet. The borings showed a bed of cemented gravel and boulders amply solid for a foundation; but, unfortunately, it was far from level, in one case there being a difference of elevation of nineteen (19) feet between the diagonally opposite corners of the caisson. Before any sinking was attempted, the foundation was prepared by blasting to receive the caisson. Holes, spaced six (6) feet centres over an area somewhat greater than that of the caisson, were put down about six (6) feet at a time into the hard material, then blasted so as to loosen the gravel; and this operation was continued for each hole until a depth exceeding by two feet that of the final position of the caisson base was attained. It was necessary first to drive a four (4) inch pipe so as to shut out the flow of silt, then to put down another three (3) inches in diameter inside of the first one. The pipes were sunk by a 3200 lb. hammer dropped from four (4) to twelve (12) inches per blow. A cross-bit drill attached to a two (2) inch pipe was used to penetrate the gravel and was followed with the case-pipe. After a penetration of about six (6) feet into the unbroken mass was reached, a thirty (30) pound charge of dynamite was lowered to the bottom of the hole, and the pipe was withdrawn to about three (3) feet above the charge, so as to be out of danger, then the explosive was fired, the casing was again driven, and the same operations were repeated until the desired depth was reached. It was found necessary to weight down the dynamite charge so as to prevent its being lifted when the pipe was pulled. A water jet was employed in the usual manner for cleaning out the pipes. During the sinking of the caissons it was often found necessary to put down holes about one foot outside of the periphery and blast the gravel into the partially excavated interior of the chamber.

Caissons to be sunk by open-dredging are usually built either of timber or of steel. The advantages of timber are generally cheapness and saving of time in procuring the materials; while the sole advantage of steel is the greater effective weight for sinking that it affords. It is, of course, conceivable that in certain localities steel would be cheaper than timber, but it is a very unlikely condition. It is much more probable that a case might arise where the metal is more readily procurable than the timber; but this, too, is unlikely.

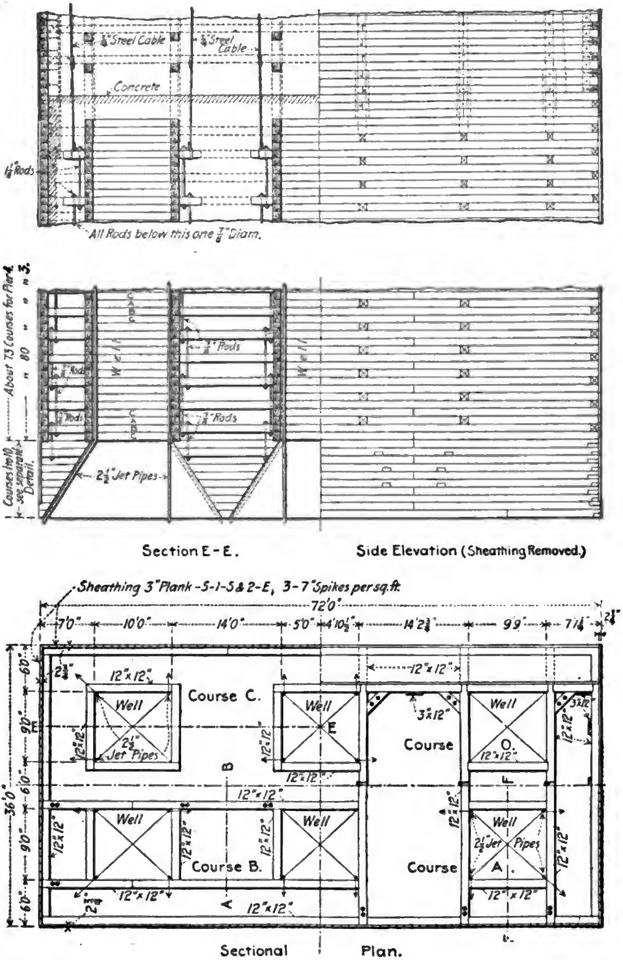

$\frac{3}{4}$" Grout
El. Top of Pier +22.25
13'0"
H.W. Line +21.0
9"
21'0"
1$\frac{1}{8}$" Eye Bolts
Tar Paper
1"×6" T.&G. Flooring
33'0"
30'0"
4"×12"×6'0" Pieces Bolted on each Side of Verticals
12'0"
3" Planking
$\frac{3}{4}$" Bolts
$\frac{3}{4}$" Steel Cable
Concrete
L.W. Line ±0.0
about 34'0"
1$\frac{1}{8}$" Rods
All Rods below this one $\frac{7}{8}$" Diam.
Section A-B.
End Elevation. (Sheathing Removed)
Well
$\frac{7}{8}$" Rods
2$\frac{1}{2}$" Jet Pipes
Section F-F.
End Elevation. (Sheathing Removed)
14'0"
$\frac{7}{8}$" Rods, 16" C. to C. Staggered
$\frac{7}{8}$" Bolts
4, $\frac{7}{8}$" Bolts
4, do.
2$\frac{1}{2}$"×$\frac{3}{8}$" Strap
$\frac{7}{8}$" Drift Bolts
30"×$\frac{5}{8}$" Plate
6"×$\frac{3}{4}$" Plate
L, 6"×6"×$\frac{3}{4}$"
Center Transverse Cutting Edge.
7'0"
$\frac{7}{8}$" Rods, 16" C. to C.
$\frac{7}{8}$" Drift Bolts
2$\frac{1}{2}$"×$\frac{3}{8}$" Strap
29"×$\frac{5}{8}$" Plate
20"×$\frac{5}{8}$" Plate
Outside Transverse Cutting Edge.
6'0"
3"×$\frac{3}{8}$"×12" Strap Splice Pl.
3"×$\frac{3}{8}$" Strap
Center Longitudinal Cutting Edge.
Outside Longitudinal Cutting Edge.

General Notes: All Timber S-4-S to uniform Dimensions. Each Timber Fastened with one Vertical Drift Bolt $\frac{3}{4}$"×30"-3'0" C. to C. All $\frac{7}{8}$" Drift Bolts Rivet Head one End and other End plain. All other Drift Bolts with both Ends plain.

FIG. 40a. Open-dredging Caisson and Cofferdam for the O.-W. R. R. & N. Co.'s Bridge over the Willamette River at Portland, Ore.

As for the question of durability, any sound timber (with possibly a few exceptions, such as cottonwood) continuously submerged in fresh water will last forever, and steel in like conditions corrodes very slowly.

In salt water the timber is liable to be destroyed by sea-worms; and, therefore, the caissons should be so designed that the destruction of the wood will not injure their strength This can be accomplished by sinking the tops of the wooden decks so far below mud-line that the greatest



Section E-E.

Side Elevation (Sheathing Removed.)

Sectional Plan.

FIG. 40b. Open-dredging Caisson and Cofferdam for the O.-W. R. R. & N. Co.'s Bridge over the Willamette River at Portland, Ore.

possible scour will not expose them, and by removing the bracing timbers from the crib as the concrete reaches them, although no serious damage would occur were they left in, for the concrete would arch over the open spaces left by the destroyed wood. The author adopted this method of protecting against the ravages of the *teredo* in the various bridges that he built in the City of Vancouver, British Columbia. It was also used by him in the Boca del Rio Bridge, which was built within a few yards of the Gulf of Mexico near Vera Cruz.

To protect against the corrosion of steel in caissons sunk in salt water, the detailing must be so designed that the concrete is nowhere wholly separated by the metal, thus leaving the mass practically monolithic. It will require some care to build steel caissons in this manner; but it can be done—and certainly it should be. It would be a wise precaution so to build them for sinking in fresh water also, in order that, if centuries hence the steel is seriously corroded, no disaster will result. As far as the author knows, this *desideratum* has not yet been attained.

Reinforced concrete caissons for open dredging have been successfully employed upon some of the work of the author's firm. In the case of the reinforced concrete arch across the Blue River at Fifteenth Street in Kansas City, Missouri, reinforced concrete shells were sunk to a depth of about (60) sixty feet. These shells, or caissons, were rectangular in form and were divided into three chambers by cross walls, and the lower edges of the sides were tapered to produce a cutting edge effect. About two and a half feet from the bottom a large recess or groove was moulded into the interior faces of all these walls; and when the caisson had landed, a five (5) foot deposit of concrete was placed in the bottom. This layer of concrete dovetailed into the groove just mentioned and thus became a fixed base continuous over the entire caisson. These chambers were then filled with sand to provide the additional weight and stability required. Movable forms were used in building the caisson. The walls of the form were held in place by bolts screwing into special nuts buried in the concrete close to the face. By unscrewing these bolts the form was freed, after which it could be moved upward for the next layer, the bolts being screwed into a higher set of nuts previously placed.

Modern caissons are made rectangular, square, or octagonal in cross-section, the last form being often adopted for pivot-piers of swing-bridges. There seems to be some doubt as to whether the saving in volume by the smaller section of the octagon as compared to the corresponding square is sufficient to compensate for the extra cost of framing the exterior timbers; but those who are best posted on such matters are convinced that the net saving is considerable. It is only when contractors are paid by the cubic yard that a plea is made for the use of square caissons to support pivot-piers. If they were paid by the lump sum and the choice were left to them, they would certainly adopt the octagon.

Metal caissons are generally circular in cross-section; but they are sometimes made rectangular with semi-circular ends. The latter section is objectionable because of the difficulty in stiffening it properly without cutting up the concrete too much.

In circular and octagonal caissons there is a central well for the purpose of excavation; and in the other caissons there are generally two (2), but sometimes three (3) wells. The size of any well is governed by the following considerations:

First. It should be made big enough to permit the largest dredge used to pass down and up readily.

Second. It should generally be made as small as practicable so as to obtain the largest possible weight of concrete in order to overcome the greatest friction that is likely to be encountered during the sinking.

Third. It should not be made so small that the distance between the cutting edge of the caisson and the reach of the bucket is so great as to permit the material to resist caving; because that would entail unnecessary labor and expense for jetting.

In designing all caissons there should always be made an allowance for a possible error of final position, because the shaft of the pier must be located properly, even if it be eccentric to its supporting crib and caisson. Usually an allowance of one foot all around the base of the shaft between it and the inner surface of the cofferdam will suffice, although contractors who are paid by the cubic yard often claim that there should be more leeway than this. In case of very narrow caissons it might be well to increase the amount somewhat not only for the sake of stability but also because long, narrow caissons are difficult to sink to exact position. For caissons to be sunk to great depth and through quicksand it is advisable to increase the play allowance—possibly to as much as two (2) feet clear all around; but this ought to be a maximum, if proper care is taken during sinking to prevent the caisson from getting out of position. It is true that in sinking one of the caissons of the Hawksbury River Bridge, it was landed some five (5) feet out of place in a direction transverse to the length of the superstructure, thus necessitating a corbeling of the shaft; but the trouble was due to an unnecessary flare in the metalwork near the cutting edge. There used to be an idea prevalent among bridgemen that all caissons should have their sides battered in order to facilitate the sinking; but this is a fallacy, because such a batter renders the keeping of the mass to correct position extremely difficult; consequently nowadays all cribs and caissons are built with vertical sides.

Timber caissons in most cases should be provided with steel cutting edges firmly attached to the wood; and in steel caissons the metal at the bottom should be drawn down to a blunt cutting edge. This is to enable the periphery of the bottom of the box to force its way through small logs and to push aside moderately large boulders that may be encountered during the sinking. Of course, if it is almost certain that no serious obstructions will be met with, the steel cutting edges may be omitted from timber caissons; but such omission might involve serious loss and delay.

The batter of the interior of the chamber (which corresponds to the working chamber of a pneumatic caisson) should be such that the material will be readily forced from the exterior toward the centre, where it will be reached by the dredges. Some engineers contend that the timbers should be stepped off so as to avoid the splitting effect of the wedge planes that exists where the chamber is lined with planks; but others claim that it is better to provide the planes so as to facilitate the flow of the material, and that the concrete or grouting placed below water will fail to fill the corners of the offsets where the stepping off is done. In case of caissons sunk through a great depth of material that is not liable to scour much, this tendency to split will be resisted by the exterior pressure of the earth, hence the wedge planes are there advisable; but, otherwise, it would be safer to step off the timbers, as was done in the caissons of the New Westminster Bridge.

In order to enable the water in the shafts to be pumped out after the chamber is sealed, all joints in both timber and steel caissons should be caulked, and the deck timbers should likewise be caulked or else have a coat of thick, hot pitch in all joints, both horizontal and vertical. The caulking should be done with oakum. The pitch used for the New Westminster caissons consisted of crude resin mixed with a sufficient quantity of tallow to make the mass stiff but not brittle. This mixture was melted in a large kettle and thoroughly stirred. It was applied by means of tin vessels with long spouts. These prevented the loss of the pitch and ensured its being placed exactly where needed. A layer of one foot of one-two grouting, placed in the dry on top of the timber deck before sinking is begun, aids greatly in making the deck water-tight. This was done in some of the deep piers of the New Westminster Bridge.

In building wooden caissons, if possible, all timbers should be of the full length or width of the caisson; and where this provision is not enforced, great care should be taken in the designing to provide proper strength. The timbers in adjacent courses should be broken on opposite sides of the centre line so that not more than fifty (50) per cent are cut at the same place. These breaks should, preferably, be made at points where cross-bracing is employed. Drift bolts connecting adjacent timbers should be spaced not to exceed four (4) feet centres, and often preferably three (3) feet. They should be long enough to pass entirely through the upper and nearly through the lower of the two timbers that they unite.

Fig. 40c illustrates the design of one of the caissons of the New Westminster Bridge, also the crib thereof. As can be seen from the drawing, the crib is divided into small spaces or pockets which were to be filled with concrete as the sinking proceeded. The walls of these spaces were not made continuous; for if they were, the mass of concrete would fail to be monolithic. By leaving large open spaces in the division walls, the concrete in the various pockets unites to form a continuous mass. It must be remembered that this concrete is placed in the dry and that the workmen can pack it up close to the bottoms of the dividing timbers.

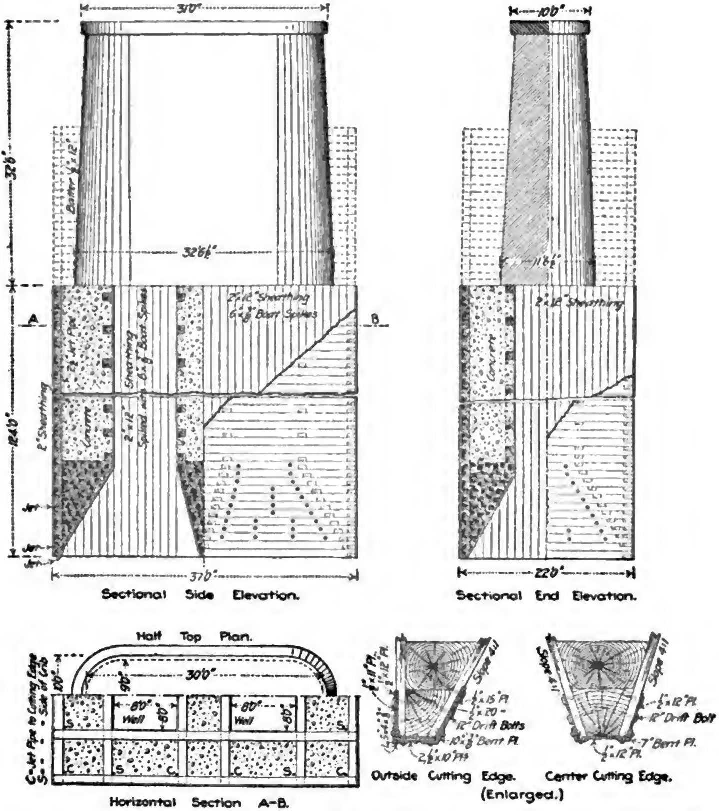


FIG. 40c. Open-dredging Caisson and Cofferdam for the New Westminster Bridge over the Fraser River.

All framing of the timber should be done in a substantial manner so that the crib and caisson will hold their shape in case that it be found necessary to force the cutting edges through logs or masses of boulders. It used to be the custom to half-dap all caisson and crib timbers at corners; but it was found better and more economical on the New Westminster

Bridge to omit the dapping and to alternate the short and the long timbers in the adjoining vertical layers. This method makes a stronger construction and saves considerable time and expense. The latest and most efficient detail adopted by the author for the corner framing is that shown in Fig. 41*a*. This is similar to that used on the New Westminster piers, except that the through timbers in each course are notched out two (2) inches at each end, and the short timbers are extended that amount so as to bear against the two-inch offset.

The author makes a practice of building pipes into the timber and concrete near the periphery of the caisson as the construction proceeds, so as to inject water for the purpose of loosening the materials. Some of his contractors have objected to these pipes on account of the extra expense which they involve and because they require extra careful caulking in order to prevent leaks around them. Very often they are not used at all; but when they are needed they are wanted badly; and their employment in even one case out of five would show a resultant economy. An incidental advantage which they possess is that they can be charged with grouting simultaneously with the chamber, and the great head of the fluid will ensure every corner thereof and every otherwise possible void in the concrete at the base being completely filled. The experiment has been made of turning some of the pipes outward through the periphery of the crib at various heights in order by jetting to reduce the side friction, but as there was no need for them, there was no proof given of their efficacy. The author believes that the expedient is a good one for cases when great frictional resistance is anticipated, especially as the cost of the pipes is small. All pipes used for jetting should ultimately be filled with grouting so as to leave no void in the construction.

Removable cofferdams of timber or steel are used to carry the sides of the cribs above water level and thus to enable the shafts to be built in the dry. They should be made as nearly water-tight as possible.

The method of constructing steel caissons for open-dredging is shown in Fig. 40*d*, which illustrates one of the deep piers of the East Omaha Bridge. The objectionable wedge effect of the conical surface of the chamber is resisted by the exterior pressure of the gravel and boulders, which must be very high at the great depth attained. In similar future constructions (especially for bridges over salt water) the author would make the roof of the chamber horizontal and support it by radial brackets, and would ensure filling all spaces by injecting grout under great head through the peripheral pipes.

Other details of construction for open-dredging cribs and caissons are given in the specifications for piers in Chapter XLII.

As soon as a caisson is launched from the ways, it is towed into proper position and held there by guide-piles properly braced, or by some other satisfactory method, then the pockets are partially filled with concrete, and the sides of the crib are built up till the cutting edge bears firmly on the bottom. Then the hydraulic elevators or dredges are put to work on excavating through the well or wells, and as fast as the material is removed the caisson sinks, the crib is built up, and the pockets are filled with concrete. These operations are continued until the required depth is reached.

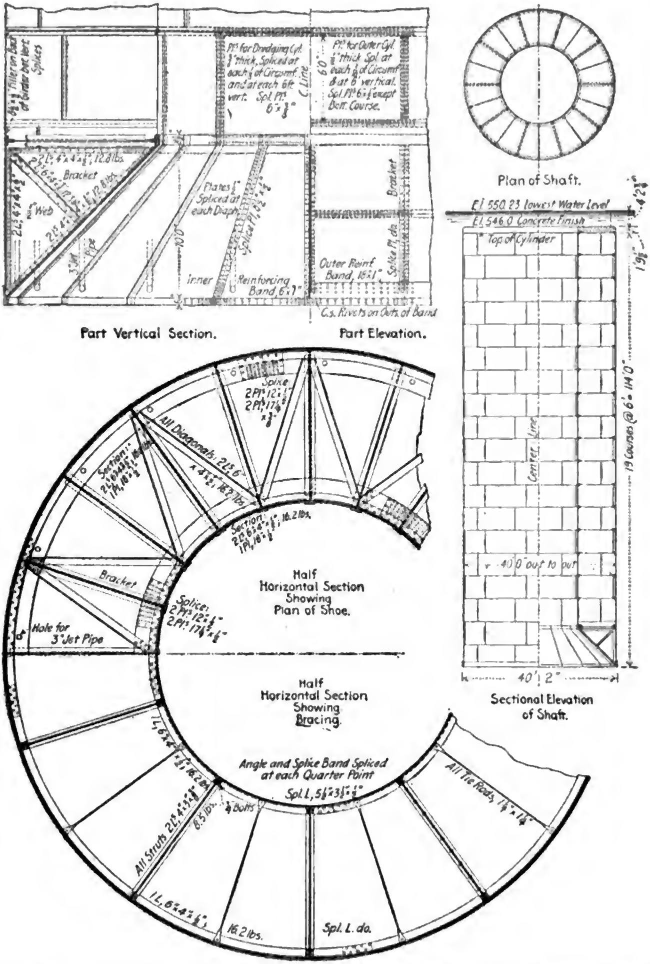

Part Vertical Section.
Part Elevation.
Plan of Shaft.
Pl. for Dredging Cyl. ⅜" thick, Spliced at each ⅛ of Circumf. and at each 6 ft. vert. Spl. Pl. 6"×⅜"
Pl. for Outer Cyl. ⅜" thick Spl. at each ⅛ of Circumf. & at 6' vertical. Spl. Pl. 6"×⅜" except Bott. Course.
C Line
6'0"
Bracket
Web
Plates ⅜" Spliced at each Diaph.
Splice Pl. 6"×⅜"
Inner
Reinforcing Band, 6"×1"
Outer Point Band, 16"×1"
Splice Pl. do.
Cs. Rivets on Outs. of Band
10'0"
El. 550.23 lowest Water Level
El. 546.0 Concrete Finish
Top of Cylinder
4'2"
19'6"
19 Courses @ 6' = 114'0"
Center Line
40'0" out to out
40'2"
Sectional Elevation of Shaft.
Splice 2 Pls. 12"×½" 2 Pls. 17¼"×⅜"
All Diagonals: 2 L 6"×4"×⅜", 16.2 lbs.
Section: 2 L 6"×4"×⅜", 16.2 lbs. 1 Pl. 18"×⅜"
Bracket
Splice: 2 Pls. 12"×½" 2 Pls. 17¼"×½"
Hole for 3" Jet Pipe
Half Horizontal Section Showing Plan of Shoe.
Half Horizontal Section Showing Bracing.
Angle and Splice Band Spliced at each Quarter Point
Spl. L, 5½"×3½"×½"
¾" Bolts
All Struts 2 L 4"×3"×⅜", 8.5 lbs.
1 L, 6"×4"×⅜", 16.2 lbs.
Spl. L. do.
All Tie Rods, 1¼"×1¼"

FIG. 40*d*. Open-dredging Caisson for the East Omaha Bridge over the Missouri River.

The best dredges for the work are the clam-shell and the orange peel. These are used for removing boulders and coarse gravel which the hydraulic elevator does not raise. There is no choice between the clam-shell and the orange peel dredges, for under varying conditions either one is likely to operate far better than the other; hence it is generally necessary on a large piece of work to have both on hand. Dredges composed of endless chains carrying buckets have been tried for sinking caissons by open-dredging, but they did not prove to be as satisfactory as either of the two types just mentioned.

The method of operation of the hydraulic elevator is as follows:

Water is forced down by a pump through a vertical pipe of about three inches inside diameter into a curved pipe which connects with a cast-steel cylinder that has another cylinder attached, the latter being connected to a perforated cast base, which is kept embedded in the material to be raised. The upper end of the main cylinder connects to a discharge pipe extending above the surface of the water and terminating in a gooseneck. The forcing of the water down the small pipe and up the large one sucks up a mixture of earth and water through the perforated casting and discharges it through the gooseneck. As long as the hydraulic elevator operates satisfactorily, it is kept going; and when it ceases to do so, it is either hoisted out of the well or shifted into one corner, and a dredge is put to work to remove the accumulation of boulders or coarse gravel that the elevator cannot carry. By thus alternating the use of the elevator and dredge great progress is made in the excavation—much more than is generally attained by the pneumatic process. Of course, the length of the hydraulic elevator has to be increased by removing the gooseneck, adding pieces of pipe, and replacing the gooseneck as the sinking proceeds.

Should the excavation go too slowly, water can be forced through the pipes that are built into the periphery of the caisson in order to loosen the material adjacent to the cutting edge and drive it toward the middle, where it will be reached by the elevators and dredges; or else a movable jet may be employed. This consists of a single pipe having at its lower end a connection with two short pipe-jets that are hinged where they join. They make an angle of one hundred and eighty (180) degrees with each other, so that the water passes out in two diametrically opposite directions; and thus the pipe is kept vertical, which it would not be if there were an unbalanced pressure, as would be the case were only one orifice provided. The hinging of these jets enables them to be shoved well over toward the cutting edges. They are moved by small wire ropes that lead along the main pipe to the top of the caisson. By rotating this apparatus slowly all portions of the cutting edge may be reached by the jets.

As the caisson approaches bed-rock it is liable to encounter a mass of overlying boulders, which tends to make the sinking very difficult. In building the East Omaha Bridge the deep caissons were forced part way through such boulders by exploding small charges of dynamite in the chamber. The resulting shock caused such vibration that at first the caisson sank several inches at each discharge, but later it lodged permanently on the boulders, which had to be left under the cutting edge, although the bed-rock was laid bare near the centre. Care must be taken not to injure the walls of the chamber by using too large charges of the explosive. Dynamiting down caissons was tried successfully also on the Koyakhai Bridge in India. On that work trouble was experienced in sinking through stiff clay, which was penetrated only by using heavy steel cutters. The author employed this expedient successfully in sinking some timber boxes for the pile piers of the Iowa Central Railway Company's bridge over the Mississippi River at Keithsburg, Ill.

The open-dredging process works well through mud, sand, gravel, and quicksand, but not so well through clay. In fact, the existence of a thick layer of very dense clay might be a sufficient reason for not adopting the process. In case of such a bed of clay overlying quicksand, if it be necessary to go into or through the latter, a combination of the two processes can be used, the pneumatic method being employed until the quicksand is reached; then by removing a diaphragm and the pneumatic shaft the caisson is made ready for the open-dredging process. The author some years ago designed two piers for a crossing of the Atchafalaya River to be sunk by the combined methods through clay and quicksand so as to reach a bed of coarse sand and gravel; but the design was rejected by the railroad company because of its apparently high cost, and the supposedly cheaper method of going down as far as practicable by the pneumatic process, landing on soft material, and protecting the piers against scour by mattresses was employed. The author protested against adopting such an inferior foundation and formally warned the company of probable disaster. It came before all the substructure work was finished, for one pier tipped forward so far that every effort made to right it and keep it plumb was unsuccessful. It was put down about one hundred and ten (110) feet below high water and could not be removed, and the company was compelled to sink a large cylinder on each side of it and to bridge across the gap so as to support the superstructure. Eventually the bridge cost far more money than the author's design would have involved, for the unit prices obtained from bidders on his plans and specific actions were not excessive, considering all the difficulties to be overcome and the unfavorable conditions for work at the crossing. The lesson to the railroad company was a severe one, because not one of its five piers can be depended upon. Any one of them is liable to fail as the first one did; and, what is worse, nothing effective can be done to forestall the trouble.

In the sinking of deep caissons one of the most important objects is to get them into true position; and the only way of accomplishing this is to start them right, guide them properly, and never let them get materially out of place. The deeper they go the easier it is to keep them to place and plumb, but the more difficult it is to correct errors of position or verticality. Some contractors in their hurry to make a great showing of progress pay but little attention to the daily position of a caisson; but the resident engineer who knows his business will never let a contractor get a caisson out of place more than two (2) or three (3) inches without insisting that the error be corrected immediately. In most cases it is practicable to build a guide-frame of timber supported by well braced piles in such a way that the caisson is held to position at two elevations, one near the water surface and the other some fifteen (15) or twenty (20) feet higher. This timber construction can generally be utilized for a working platform.

In some cases a false bottom is placed on the caisson to float it into position, and after getting it there this bottom is removed; but generally such an expedient is not needed. Often it is necessary to suspend the caisson by four (4) or more long, adjustable rods from the surrounding timber construction, and to lower it gradually until the base bears on the soil sufficiently to release all tension on the rods, after which the latter are to be removed.

In the New Westminster Bridge, the contractors, Messrs. Armstrong and Morrison of Vancouver, B. C., built a dock of long, closely-driven piles around the location of each deep caisson, leaving the down-stream end of each open until after the caisson had been floated in, when the space was closed by piling to break the force of the incoming tide. They used a novel method of bracing these piles, many of which were over one hundred (100) feet long (for the deepest water was about eighty feet), viz., by hinging a long pile to another pile at a point which would be a little above the mud-line in final position, and as the main pile was driven by combined hammer and water-jets they let the bracing pile drop gradually up or down stream, supporting it temporarily by ropes, and finally bolting it above water-level to other piles. Some wire ropes also were used for diagonal bracing, being fastened to a front pile near the mud-line and to a rear pile near its top.

The principal obstacles encountered in sinking caissons are large boulders, logs, and hard clay or compacted material. If the boulders are in the interior of the caisson and not too large, they can be removed by the dredge, but if they come under the cutting edge, they have to be jetted until they are either pushed outside by the weight of the caisson or fall inside. In case of boulders too large to be lifted through the well or handled by the dredges or other available apparatus, they may have to be broken with small charges of dynamite and removed in pieces, a diver being sent down to attend to the drilling and the placing of the charges. A specially designed compressed air drill in the hands of an experienced diver will soon make a hole of the required depth. Otherwise, if the boulder be encountered near the level of the final position of the chamber, it may be washed down and left in; but in such a case the filling of the base of the chamber for at least one-half the depth of the boulder must be of grouting so as to preclude the existence of voids beneath the obstruction. Logs lying across the cutting edges often give considerable trouble. If they are too strong to be broken by the weight of the caisson, they have to be shattered by dynamite and removed in pieces. The greatest obstacle to sinking by the open-dredging process is a bed of hard clay or other compact material. If the layer be comparatively thin, the caisson can be worried through it with water-jets in the hands of a diver; but if it be thick, it may become necessary to put on air pressure temporarily until the hard layer is passed; and unless proper provision was made at the outset for so doing, the job will be found quite difficult, especially if the depth of the obstruction below water be great.

If much trouble is anticipated from scour, it is often best to build a large, strong mattress of willows over the pier site before the sinking of the caisson is started. Some engineers and contractors in such cases figure on cutting out the mattress beneath the base of the caisson; but the author prefers to weave a hole in it with a margin of about three (3) feet all around, in order to provide for a possible error of position in placing the mat.

After a caisson has reached its final position and has been cleaned out as much as deemed necessary, it should be sealed by depositing through a trémie some two or three feet of Portland cement grouting, mixed in the proportion of one part of cement to two (2) parts of sand. This should be followed by a very rich concrete of gravel or small broken stone, mixed at first in the proportion of one, two, and one and a half, and gradually thinned to one, two, and three. This concrete should be deposited by either a trémie or a water-tight collapsible bucket, and should be carried to such a height as will, in the opinion of the engineer, effectively seal the interior against water after the well or wells have been pumped out. In some cases ten (10) feet of such sealing might suffice, but in others twenty (20) or thirty (30) feet might have to be used. In very deep foundations it would be necessary to fill more than enough for mere sealing, as it would not be advisable to pump more than a certain distance below water level. At New Westminster the greatest depth pumped out was seventy-five (75) feet below the surface of the river, and, in the author's opinion, this is pretty near the advisable limit, although in this case there was no leakage at all through the bottom and very little indeed through the sides. Of course, if much leakage be found during the pumping, the shafts can be filled again with water and more concrete can be deposited through it until a safe height is reached; but such attempts involve loss of time, as the concrete should be allowed to set a week or ten (10) days before the pumping out is begun. If the amount of concrete placed through the water be too small, or if the time of setting be too short, when the pumping is done the pressure from below may either break up the mass or force water through it, thus destroying its continuity and doing great damage.

After the water is all removed from the wells, the remaining space is to be filled in the dry with ordinary one-three-five concrete, tamped wherever necessary to fill corners and small spaces. Before beginning to fill the shafts, the linings thereof should be torn out as low as it is possible to remove them without permitting too great a leakage. This is to ensure a proper bond between the concrete filling of the shaft and that of the pockets.

What the limit of depth is which can be reached by open-dredging through sand and gravel is hard to anticipate. It is probably considerably in excess of two hundred (200) feet below water with a penetration of over one hundred (100) feet into the earth. The author once made a design for open-dredging caissons for such a depth, and stated to his prospective client that he anticipated no difficulty in sinking them; but he had no opportunity to prove the correctness of his statement, as the project did not materialize.

The side friction on caissons is known to vary generally from four hundred (400) to six hundred (600) pounds per square foot of area, and the weight of the mass is easily calculated; hence for any particular case it is practicable to compute about how far a caisson can be sunk by its own weight and how much further by a reasonably large temporary loading. Such a loading can be applied by building a timber platform, cantilevered beyond the crib so as not to interfere with the sinking, and loading it with pig-iron, stone, or even sand; but it is only when a caisson is near its final position that such an expedient should be attempted, because it would be impracticable to add to the height of the crib after the loading platform is placed—besides, the temporary loading should never be dropped as low as the water level.

If a caisson cannot be driven below a certain depth by all the ordinary means of sinking, it is almost certain to have reached a safe position; and if any doubt be felt about the matter, it is easy to provide an effective protection of mattress-work, rip-rap, or both. In extreme cases, where there is any uncertainty about the lateral stability of the finished pier, the mass of rip-rap can be carried up to such a height as to make it absolutely safe against overturning in any direction. This was done in the case of one of the large piers of the New Westminster Bridge that landed on a mass of cemented gravel which could not be penetrated by the dredges, and which was too far below water to warrant applying compressed air.

If a caisson is hung mainly by side friction, and it is desired to sink it a little further, it may be easier to scour around the outside by powerful water jets than to put on a temporary loading.

If a caisson becomes tipped, it may be best to block it temporarily by short timber posts on the low side, placed by a diver, then continue the excavation until it is righted before removing the blocking; or it may be necessary to excavate on the exterior along the high side until the cutting edge is undermined. This had to be done on the pier of the New Westminster Bridge, to which reference has just been made. It must be remembered that after a caisson has attained considerable penetration through good, firm material, such as sand or gravel, if it strike a harder bearing on one side than on the other, it will not tip much, because it will be held firmly by the earth above; hence it is usually only with small penetrations that difficulty from tipping is encountered, unless, perchance, a sliding bank of gumbo or clay be met, as was the case at the Atchafalaya River Bridge before mentioned.

Usually the neatwork of a pier is started at an elevation of two (2) or three (3) feet below extreme low water level, but in some cases it is deeper than that. In order that the said neatwork may be laid in the dry, which is absolutely essential, the crib will have to be continued above the greatest possible water level that will be encountered during the construction of the shaft by means of removable timbers caulked so as to form a water-tight cofferdam. Exactly the same construction is required for the cribs of pneumatic caissons.

The illustrations for this chapter are the same as some of those used in Messrs. Jacoby and Davis' treatise on "Foundations of Bridges and Buildings," published by the McGraw-Hill Book Company; and to all these gentlemen are due the author's thanks not only for their kind permission to reproduce the said drawings of three of his open-dredging structures, but also for their courtesy in furnishing him with electrotypes of the cuts.

CHAPTER XLI

PNEUMATIC PROCESS

THE pneumatic process for securing pier foundations is best suited for depths between thirty and one hundred feet. In most cases it is the best method of sinking to employ, the greatest objection to it being the excessive cost of installing the plant, even if one has a complete outfit at his disposal.

The advantages of the pneumatic process are as follows:

First. It enables the contractor to overcome, in the cheapest and most expeditious manner possible, all obstacles that may be encountered during sinking.

Second. It ensures the obtaining of a satisfactory foundation for the piers, because any unevenness in the bearing can be leveled off, and the caisson can be sunk into bed-rock a few feet, thereby securing an effective anchorage.

Third. It permits of the working chamber being filled with concrete placed in the dry, thus ensuring a better grade of concrete because of freedom from laitance.

The disadvantages of the pneumatic process are as follows:

First. Expensive pneumatic machinery to purchase, to ship to and fro, and to keep in repair.

Second. High priced labor.

Third. Danger to workmen from compressed air.

Fourth. Less daily progress in sinking, due both to the upward pressure of the air in the working chamber and to the necessity for locking out the coarse materials excavated.

Fifth. Less attainable depth as compared with the open-dredging process, because of man's inability to work under pressures exceeding that of the atmosphere by fifty pounds per square inch.

However, notwithstanding these disadvantages, it is probably the most satisfactory, all around method in nine cases out of ten of important bridgework which occur in a consulting-engineer's practice. Most of the piers of the large bridges which the author has engineered have been sunk by the pneumatic process; and he has no hesitation in recommending it for conditions within the above specified limits.

The distinctive feature of the process is the employment of a working chamber in the caisson and excluding the water by compressed air, thereby enabling the workmen to enter the chamber and excavate the material necessary to sink the caisson. The latter is open at the bottom and closed at the top with a tight, heavy roof. Above this roof is the crib filled with concrete and extending up nearly to low water level. For the ingress and egress of workmen and of large material, two or more vertical shafts are provided. In these are placed air locks so that the transition from working pressure to normal pressure, or vice versa, can be made gradually and with small waste of air and small change of pressure in the working chamber. This pressure is usually made to equal approximately a half pound for each foot of penetration below the water line. It adds to both the comfort and the efficiency of the workmen to cool the air in the summer time and to warm it in the winter.

For the ejection of fine material, a vertical four-inch pipe is run from the working chamber up through the mass of the roof and crib to some convenient height above the water line. On the top of this pipe is the "goose-neck," which is a heavy curved casting for directing the discharge away from the crib. In the working chamber the blow pipe is carried down so that the material can readily be kept in a heap about its mouth. When the blow pipe is opened up, the air pressure carries the adjacent material through the pipe, discharging it from the "goose-neck." A duplicate blow out pipe should be installed so as to be ready for use, if, for any reason, the first one becomes stopped up and put out of commission.

The bottom of the caisson is shod with a steel cutting edge usually made of plates and angles riveted together. This enables the caisson to cut through small logs and to push aside boulders, and lessens the resistance to sinking because of its small bearing area.

The building of the caisson may be started on shore, or on a frame work supported by piles driven around the site of the pier. The choice of these two methods will depend largely upon the convenience of getting material to the place, the danger of sudden rises in the river, and the degree of menace from navigation. If the caisson construction be started on shore, the cutting edge is assembled on timber shoes resting on ways arranged for convenient launching. The timber work is then built up to a convenient height, and the shoes are released so that the mass slides down the ways into the water. It is then towed out to the pier site, where guide piles have previously been driven, placed in approximate position, and secured to the guides. Here additional weight is added by filling in the crib with concrete and increasing the height. As the caisson approaches the bottom, it is carefully watched and corrected for position. It is allowed to sink into the soft material of the river bed, and then the air is turned on and further sinking is obtained by excavating within the working chamber. In case the river bed is too uneven to afford a good landing for the caisson, it is a good plan to dump sacks of sand into the low places until a level bed for its reception is produced.

For streams where low water obtains for a sufficient time and where navigation does not interfere, it is economical to build a temporary pile trestle or runway from the shore to the pier site and start the caisson at the final location. For this purpose piles are driven around the pier site and capped, then cross timbers are laid on top, upon which timbers the cutting edge is assembled and the caisson built. Several gallows frames are erected on the caps, and, by means of long, heavy screws for hangers, the caisson can be suspended when so desired and the supporting timbers removed. Then by turning the nuts on the screws, the mass can gradually be lowered to the river bed as fast as the crib is built up and filled with concrete. After landing firmly on the bottom the gallows frames are to be removed.

During the early part of the sinking, say the first twenty feet below water, the material may advantageously be blown out dry. After this depth is reached, the air pressure becomes too great for the loose, dry material; and excessive leakage occurs through the voids, so that it becomes more economical to use a wet suction. For this purpose a two-inch pipe is run into the caisson from the crib above in the same manner as was the blow pipe. This small pipe is connected with the pump to supply a stream of water. A hose is attached to the pipe, and by means of a one-inch nozzle a good water-jet is obtained. This water-jet is employed for wetting down the material, so that it will pass through the blow pipe with less friction, and will fill the cross section thereof better, thus preventing excessive loss of air. Where clay is encountered, it must be broken up into small pieces and thoroughly saturated so as to move freely through the blow pipe. Another advantage of blowing wet materials is that it does not wear out so quickly the "goose neck" casting at the top of the blow pipe. The wet material is conveyed to the blow pipe by means of a four (4) inch flexible hose having a three and a half (3½) inch suction nozzle. This hose is attached to the blow out pipe; and it should be long enough to reach all parts of the working chamber, as it is much easier to move the flexible hose than to shovel the material to it. The object of the three and a half inch suction nozzle is to insure that anything passing the smaller opening will be carried through the larger pipe without plugging it.

In case the location of a pier is on the bank or on a sand bar in the river, the excavated material will have to be hoisted out until the cutting edge of the caisson gets down to the water, when the air can be turned on. However, the pressure will hardly be sufficient at such a depth to force the material up the regular blow pipe. In such a case it is well to bore a hole in the side of the caisson above the water line and put the blow pipe through it. This will reduce the vertical lift so that the pressure will then be sufficient to discharge the material. When the caisson sinks, the pipe can be withdrawn and the hole in the side plugged up. By this time the working chamber will hold the air well enough to raise the material through the vertical pipe.

Another scheme, in case of a pier located on the bank or on a sand-bar, is to excavate a large hole down to the level of the water and start the construction of the caisson therein. There are two good reasons for so doing, viz., first, it is usually much cheaper to excavate in the open a large quantity of earth by scrapers or dredge than to take out a small quantity by blowing or hoisting through the caisson; and, second, the side friction is thus reduced, enabling the caisson to descend more readily to its final position.

As the mass lowers, the crib is carried up and filled with concrete from time to time so as to put as much load on it as possible in order to facilitate the sinking. The supply shaft, man shaft, blow pipes, and water pipes are carried up likewise. As the greater depths are reached, the skin friction of the material surrounding the caisson and crib becomes excessive, and in conjunction with the air pressure holds the sinking in check. To overcome this the excavation is carried below the cutting edge as far as practicable, then the air supply is shut off and the air in the working chamber is suddenly released so that an impact is given to the mass, and a flow of water is set up along the sides and under the cutting edge, thus reducing the friction and causing the material at the sides of the excavation to cave in and permit the caisson to settle into a new position. This process is termed blowing the caisson. It is during this act of blowing that the mass is most likely to get out of true position, for variations in skin friction may hold up one side more than the other. As long as it is practicable to add to the caisson sufficient weight to cause it to settle gradually into position, blowing should be avoided, because there is far less chance of its departing from true position when the sinking is continuous and gradual.

The facility with which a caisson can be kept in correct position during the sinking depends greatly upon its width and the verticality of its sides. A narrow caisson will invariably work from position, unless there are several guide piles and ample waling on each side to hold it in place. These piles should be located three or four feet from the periphery and driven to a depth of thirty or forty feet, then thoroughly braced and waled, after which blocking is to be inserted between the waling and the caisson. When the latter reaches a depth of forty feet below the river bed, there is small danger of its working to any important extent from true position, yet the author believes that in many cases it would pay to make caissons somewhat wider than the ordinary minimum of fourteen or sixteen feet.

In times long past when the pneumatic process was in its infancy, substructure designers used to think it was necessary to flare the sides of the crib and caisson in order to reduce the side friction; but this idea was a fallacy, because such flaring kept the descending mass in a state of unstable equilibrium and often caused great variations from correctness of pier positions. As stated previously, a glaring case of this occurred when building the great Hawksbury Bridge in Australia.

On the Illinois Central Bridge at Omaha, Nebraska, which work was engineered by the author's firm, considerable trouble was had with two pneumatic caissons. One was sunk on land, and when within twenty feet of the final position it was tilted and shifted out thirty-two inches at the bottom. After working a long time and using every known means to bring it back to position, during which the situation became more aggravated, it was finally decided, as a last resort, to put a wire cable around the cribbing and concrete and anchor the same to two dead-men on shore. This was done and the cable was tightened as much as possible with blocks and tackle acting as a toggle. The stress on this apparatus was maintained during the rest of the sinking; and due to it the top of the cribbing was pulled shoreward, thereby tipping the bottom and getting it back to the proper place. When the caisson landed it was only one-half inch off centre. In sinking the second caisson for the same bridge, an old mattress and some riprap, which had been placed around a pile pier for its protection, were encountered. This condition caused considerable trouble, as the riprap followed the caisson down, causing it to depart thirty-six inches from the correct position, but by means of a toggle similar to that used in the first case it was righted to within four inches of its true position. Good anchorage for the toggle was not available in this second case, hence the method was not as successful as in the first instance.

One of the more customary expedients for rectifying small departures from correct position is to load the high side on top of the crib, in case of a lean, and to deposit the material from the blow pipe in the river on the low side, thus putting a surcharge on the river bed there, increasing the skin friction and holding up that side of the caisson. At the same time the excavation in the working chamber is confined to the high side; and in aggravated cases it may be extended out beyond the cutting edge so as to induce caving and a lessening of skin friction on that side. This loosening of material can also be helped by running a water jet under the cutting edge and directing the stream upward or by jetting the river bed along the high side of the caisson. Sometimes it is advantageous to introduce mudsills into the working chamber, placing them along the low side of the caisson, then to put heavy timber props on the mudsills and to wedge in between them and the roof of the working chamber. By blowing the caisson a sinking may be obtained on the high side while the low one retains its position because of the props. This method is not suitable if the mudsills rest on material that readily scours; for the influx of water occurring on the rapid falling off of air pressure will wash the material from around and under the sills so that their supporting value becomes nil.

It sometimes happens that the caisson moves bodily out of line, and then a tilting that will direct it back to correct position must be given it, although for the time being such tilting may cause the top to move more out of line than before. As the sinking progresses with the caisson thus directed, the bottom will approach its true position, and then the top can gradually be brought back to line and the caisson made upright.

During the sinking process, obstructions such as logs, boulders, flat, detached slabs of limestone, or thin strata of sandstone or limestone are sometimes encountered. These have to be broken up by chopping, sawing, or blasting into small enough pieces to permit of their being hoisted through the shaft. In the case of the Sioux City Bridge, soft, friable sandstone was met with where the wash-borings had indicated only sand. This had to be blasted and removed in small pieces until a suitable bed had been made on which to land the caisson. In sinking the caissons for the piers of the Missouri River Bridge at Jefferson City, large detached blocks of limestone that in some previous age had broken off the rock ledge and slid into the river channel were encountered. These had to be blasted into small pieces and taken out through the supply shaft, making a slow and costly job of excavation and pier sinking. However, these were finally removed, and the caisson was carried down below the layer of blocks. An unusual obstruction was met with in sinking one of the caissons for the Kaw River Bridge of the Intercity Viaduct at Kansas City, viz., a thirty-inch, high-service water-main lying so close to the pier site that the caisson had to be rotated a little to clear it. Pile bents were driven under the pipe for support, but these went down as the caisson sank, consequently a sling was put around the pipe and passed up over an A-frame and run to a distant anchorage. It was thought that the caisson might be swung back to its true position after dropping below the pipe, but all efforts to rotate it were unsuccessful.

While much ingenuity and resourcefulness are demanded of the constructor in pneumatic foundation work, it must not be inferred that all jobs present the difficulties which have just been cited. The author has used the pneumatic process on many other bridges where comparatively little trouble was met with, such as the Red River bridges at Index, Tex., and Alexandria, La.; the Missouri River Bridge at St. Charles, Mo.; the Arkansas River Bridge at Fort Smith; the I. & G. N. Railway Bridge across the Brazos in Texas; three bridges over False Creek for the City of Vancouver, at Westminster Avenue, Granville Street, and Cambie Street; two bridges for the Canadian Northern over the Thompson River; two bridges for the Great Northern Railway over the Missouri and Yellowstone Rivers; the Arkansas River Bridge at Pine Bluff for the Cotton belt Railroad; the Halsted Street lift bridge at Chicago; the bridge over the Willamette River at Harrisburg, Ore., for the Oregon Electric Railway Co.; and others.

After the caisson has landed properly and the bed-rock is cleaned off, the working chamber is filled with concrete, and allowed to set from twenty-four to thirty-six hours, then the air is cut off and the air-locks are removed, after which the shafts are filled with concrete. The timber crib is extended above the water line so as to act as a cofferdam while the shaft of the pier is being started. This shaft should be begun as near to correct position as possible, even if some eccentricity of loading on the caisson results.

The design of pneumatic caissons has been developed largely through the accumulation of experience. The wooden caisson has reached a rather high state of development, but it is likely that the increasing price of timber will necessitate the adoption of steel caissons and, possibly, reinforced-concrete ones. While attempts have been made to develop a rational design for the latter type, there is need for further study and investigation along that line. The forces to be considered are the vertical loads, skin friction on sides, lateral earth thrusts, and concentrated reactions at one or more points on the cutting edge due to logs, boulders, and other obstructions. Twisting and warping occur and complicate the analysis of stresses. The walls have to resist torsion in addition to bending moments and shears.

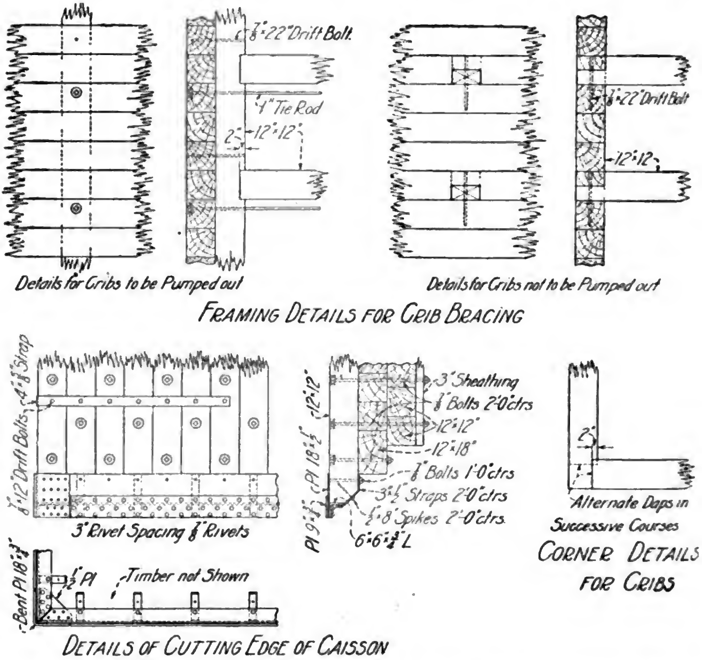


FIG. 41a. Details of Cutting Edge and Framing of Crib for a Pneumatic Pier.

The advantages of the concrete caisson are the increased weight per volume, avoidance of caulking, freedom from leakage for either air r, and indestructibility. The disadvantages are the greater skin and the longer time required in sinking. This latter is due to that, as the crib is built up from time to time, the concrete must enough after each operation to provide the strength required

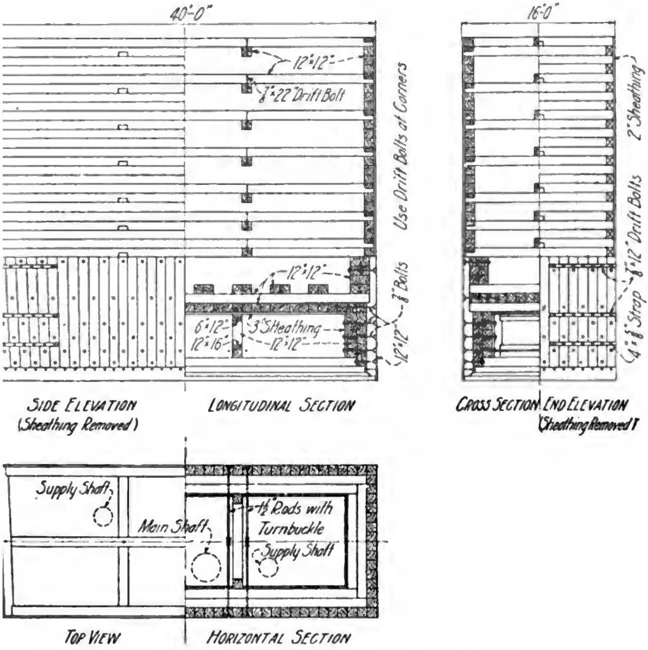


Fig. 41*b*. Details of Pneumatic Caisson for a Rectangular Pier.

esist the stresses produced by the distortions during further sinking. umber of all-concrete caissons have been constructed in the past few rs, and it seems likely that they will be used more extensively in the re.

In many of the recently built pneumatic caissons, the timber roof has n replaced to a large extent with concrete. This gives a tighter roof, l it has less compressibility than one composed of a number of layers 12″ × 12″ timbers, besides affording additional weight for sinking.

The air-locks should be placed at the top of the shafts, where they always accessible and beyond the danger of flooding. A single door to be provided at the bottom of each shaft, which door can be closed ile the air-lock is being removed for the purpose of adding new sections to the shaft. The air-locks should, preferably, be operated from the inside. The main shaft is usually three feet in diameter, with steel rounds built in so as to form a ladder. The connecting angles between sections should, preferably, be turned inward, so that, if any leakage develops, the

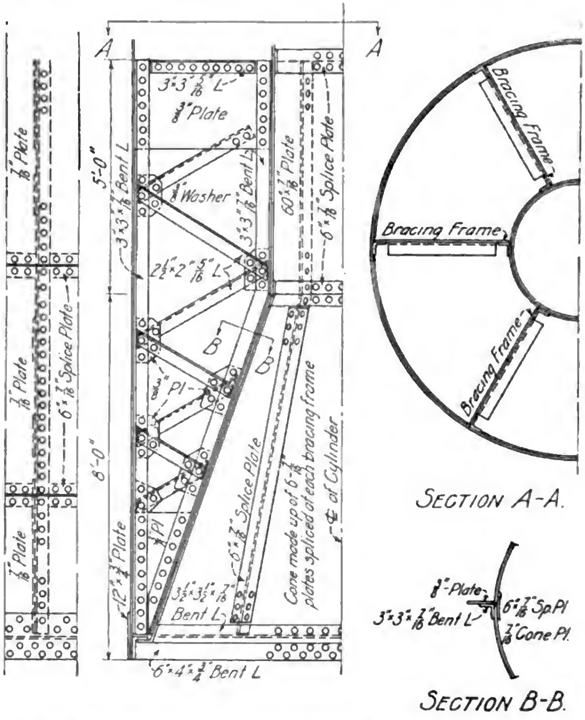

A
A
$\frac{7}{16}$" Plate
$\frac{7}{16}$" Plate
6"×$\frac{7}{16}$" Splice Plate
$\frac{7}{16}$" Plate
5'-0"
8'-0"
3"×3"×$\frac{5}{16}$" L
$\frac{3}{8}$" Plate
3"×3"×$\frac{7}{16}$" Bent L
$\frac{3}{4}$" Washer
3"×3"×$\frac{7}{16}$" Bent L
60"×$\frac{7}{16}$" Plate
6"×$\frac{7}{16}$" Splice Plate
2½"×2"×$\frac{5}{16}$" L
B
B
$\frac{3}{8}$" Pl
$\frac{3}{8}$" Pl
12"×$\frac{3}{4}$" Plate
3½"×3½"×$\frac{7}{16}$" Bent L
6"×$\frac{7}{16}$" Splice Plate
Cone made up of 6-$\frac{7}{16}$" plates spliced at each bracing frame
℄ of Cylinder
6"×4"×$\frac{3}{4}$" Bent L
Bracing Frame
Bracing Frame
Bracing Frame
SECTION A-A.
$\frac{3}{8}$" Plate
3"×3"×$\frac{7}{16}$" Bent L
6"×$\frac{7}{16}$" Sp. Pl
$\frac{7}{16}$" Cone Pl.
SECTION B-B.

FIG. 41*c*. Details of Pneumatic Caisson for a Steel Cylinder Pier.

bolts can be tightened. Also it may be desirable to remove the sections, and then the bolts can be gotten at readily. The supply shaft is generally made about two feet in diameter, and is arranged so that a bucket for hoisting the coarse material excavated or for lowering the concrete can be run through. The Moran air-lock is worked in an oval shaft, and provides for the bucket occupying one side and the men the other. The operating rope slides through a stuffing box so that very little air escapes. The disadvantage of this arrangement is that it takes longer to lock the material in and out if men are passing through at the same time.

Figs. 41*a* and 41*b* show the details of a timber caisson and crib for a rectangular pier; while Fig. 41*c* gives the details of a pneumatic caisson for a steel cylinder pier. The details in general conform to those for open dredging piers. In Chapter XLIII will be found specifications governing their design.

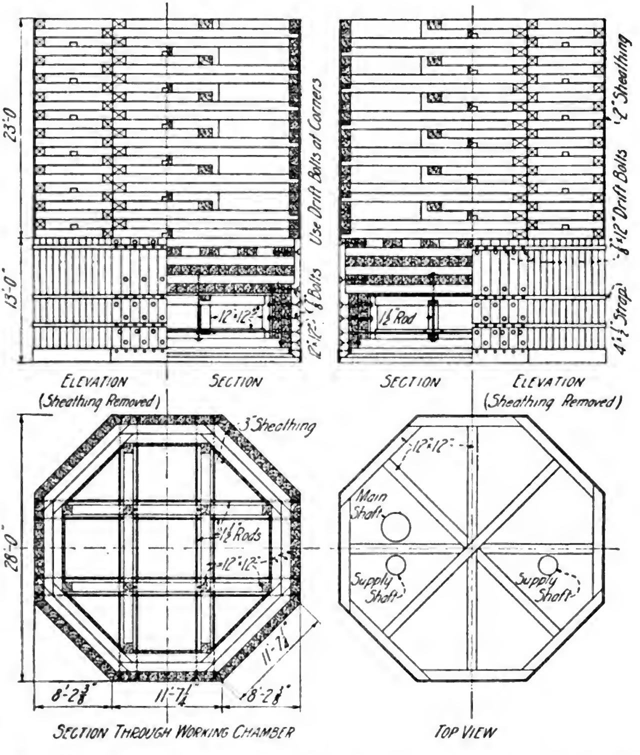


FIG. 41*d*. Details of Pneumatic Caisson for Pivot-pier of the Granville Street Bridge over False Creek in Vancouver, B. C.

Fig. 41*d* illustrates the pneumatic pier for the draw-span of the Granville Street Bridge over False Creek in Vancouver, B. C., which was designed by the author's firm.

CHAPTER XLII

PILES AND PILE DRIVING

As there is already a large amount of literature extant on this subject, only the more salient and important points will be referred to in this chapter, leaving the reader who wishes to study details to consult such works as that of Jacoby and Davis on "Foundations of Bridges and Buildings." There is an extensive bibliography of the matter published in Volume 10, Part 1, of the *Proceedings* of the American Railway Engineering Association. Commencing at page 579 therein, this bibliography covers fourteen pages that embrace some three hundred articles on all features of the subject.

The customary objects in using piles are to compact the soil, to carry vertical loads, to resist horizontal pressures, and to enclose openings. To fulfil these various functions, different kinds of piles are employed. The round, square, or octagonal-shaped ones are best suited for the first three purposes; while the flat or sheet pile is best adapted for the last.

Soils containing large percentages of clay can be compacted to a considerable depth by driving numerous small, short piles closely together over the desired area, then pulling them one at a time and filling the holes with fine, sharp, clean sand of uniform size. If the sand be rammed in thin layers as it is deposited in the hole, lateral pressures will be set up as the load is applied, and the resulting arch action will transmit a portion of the load to the sides of the filled hole.

Bearing piles, supporting superimposed loads, depend usually for their carrying capacity upon the friction of the material which they penetrate. In some instances, though, the ends reach or enter a hard stratum; and under such conditions the piles act as columns, and their bearing capacity is limited only by their strength in compression.

In making substructure designs it is generally necessary to assume the total permissible load per pile, based upon the rather meagre information concerning the character of the material to be penetrated that is available before construction is begun. In a few important cases test piles have been driven in advance of the preparation of the substructure plans; but, unfortunately, the client usually objects to the expense that would be involved by such unusual testing; consequently the designer nearly always has to use his best judgment concerning this important matter. It has been the author's practice to assume, when the effect of impact is ignored, a load per pile varying from twenty (20) to thirty (30) tons, and usually about twenty-five (25) tons; although in certain cases of highway bridges on the Pacific Coast, where the piles were large in diameter and exceedingly long, where the material to be penetrated was fairly hard and perfectly safe from scour, and where the greatest legitimate economy in first cost was a necessity, the limit has been raised to forty (40) tons. On the other hand, though, in certain cases when the piles were short, or when the material to be penetrated was not firm, the load has been reduced to ten (10) tons per pile.

In settling this matter of pile loading, consideration should be given to the question of the probable occurrence of the maximum load and its frequency of application, adopting a higher unit for improbable and unusual loads than for certain and oft-repeated ones. For instance, in the piers supporting the towers of high vertical-lift bridges, the effect of wind pressure on the pile loading is comparatively large, but its maximum assumed effect may never exist, and certainly not when there is much live load on the structure; hence for such conditions a large unit loading is permissible.

Sometimes the engineer may have the opportunity to watch the driving of a test pile and thus be able to note the amount of penetration gained from the last blow of a hammer of known weight and drop. From these data he can estimate the carrying capacity of the pile by using some such formulæ as Wellington's, known generally as the *Engineering News* Formulæ.

They are the following:

For a pile driven with a drop hammer,

$$P = \frac{2Wh}{s+1},$$

and for a pile driven with a steam hammer,

$$P = \frac{2Wh}{s+0.1},$$

in which P is the safe load in pounds, W the weight of the hammer in pounds, h the fall of the hammer in feet, and s the penetration or sinking in inches under the last blow, assumed to be sensible and at an approximately uniform rate. The said sinking must be measured only when there is no visible rebound of the hammer and only when the last blow is struck upon practically sound wood.

In making an observation of this kind, care should be taken to see that the hammer has a free and moderate fall of about fifteen feet for one weighing three thousand pounds; to ensure that any brooming of the top of the pile is previously sawed off, so that there will be solid wood to receive the blow; to make certain that the hammer rebounds very little, if at all; and to beware of using too small a penetration. Nothing under a half inch should be considered; and an inch will give more reliable results. At the best, the knowledge thus obtained is of a relative order only, as will be readily appreciated when comparing the results of the tests with those of actual loading experiments. There are on record and convenient for reference a number of reliable tests of actual ultimate loads carried by piles under different conditions. The standard engineering handbooks give tables which will serve as a guide to one's judgment; but in any important work where soil conditions render uncertain the action of the pile, a careful test should be made by driving several piles and loading them until they fail. Then a factor of safety of two should be applied to these ultimate loads in order to obtain safe loads for designing purposes.

Much effort has been spent in trying to evolve a general formula which would include all the variables and give consistent results for all conditions. Two methods of attack have been pursued. The dynamic method, by which it is attempted to account for the energy of the falling hammer in doing work on the pile, is best treated by E. P. Goodrich, Esq., C. E., in the *Transactions* of the American Society of Civil Engineers, Vol. XLVIII. The static method, by which it is attempted to determine the bearing power of the pile by applying the principles of earth pressures, has recently been discussed by T. C. Desmond, Esq., C. E., in Vol. LXV, and by J. H. Griffith, Esq., C. E., in Vol. LXX of the *Transactions* of the American Society of Civil Engineers. These investigations are valuable in giving one a keener appreciation of the complexity of the phenomena involved, as well as for advancing the problem toward its ultimate solution. They also serve to emphasize the need of caution in applying formulæ without a knowledge of their derivation and limitations. It should be recognized that different soils exhibit in different degrees the physical properties of elasticity, compressibility, viscosity, and internal friction. It is too much to expect a simple formula containing two or three variables only to give consistent results for the various combinations of the many factors involved. The dynamic method at best can only give the bearing power at the time of driving, while the static method applies to any period after the regaining of equilibrium by the disturbed soil. The time required for such restoration of equilibrium varies from a half hour to twelve hours. Instances are on record where piles have been easily driven in soft soils, and then allowed to rest for several hours; and on resuming driving some difficulty was experienced in starting them again, requiring several blows of the hammer before a perceptible motion would result. It has been found to be generally true that the resistance of piles penetrating soft material, and depending, therefore, solely upon skin friction, is substantially increased after a period of rest. It has also been found that such piles have their bearing capacity reduced by driving additional piles too close to them. The minimum spacing should not be less than three feet.

It is sometimes necessary to use piles for resisting horizontal pressures as in dykes, wharfs, fenders, guide piles, and even abutments of arches. The resistance that a pile, or a group of piles, can offer to a horizontal force becomes at times an important matter. Little has been done to investigate this feature; and the engineer will find scarcely any information concerning it, upon which to base his judgment. While the strength of the pile as a beam can be estimated with reasonable accuracy, the amount of penetration required to develop that strength is not so well established. That the lower end of the pile receives considerable restraint, even in soft soil, is shown by the fact that piles driven in the plastic material of the Hudson River bed have broken near the mud line when overstressed by side pressure at the top.

When piles are placed in clusters, their resistance to horizontal pressure can be increased by connecting them with substantial sway bracing which tends to fix the upper ends and to reduce the bending moments upon them. The bottom horizontal brace should be placed as near the ground line as possible. The action of the sway bracing is similar to that of the diagonal bracing in a portal for a through bridge span. Where pile heads are encased in a mass of concrete reaching to the ground line, a high degree of fixedness is obtained, and consequently there is a great reduction in the bending moment on the piles, which then resist the horizontal pressure largely by shear. An example of this kind is seen in the abutment of an arch when the foundations are reinforced by piling. Another method of resisting horizontal forces is to drive the piles with a batter. This is frequently done for retaining wall foundations, arch abutments, and outside piles of high trestles. Piles can be readily driven with a batter of one in five.

For enclosing openings, as in cofferdam construction, sheet piles are extensively used. This is especially true since the development of the interlocking steel sheet pile, which has proved a very effective means of shutting out water, shoring up trench walls, and resisting lateral pressures.

The use to which a pile is to be put largely determines the type or kind to adopt. Piles for carrying vertical loads may be of the ordinary plain, the pedestal, the screw, or the lagged type. The plain pile may be of several kinds of timber, such as oak, long leaf yellow pine, cedar, cypress, Douglas fir, or tamarack; or it may be made of concrete with or without steel reinforcement. The plain pile is adapted for soft soils where skin friction develops a "conoid of pressure" that transfers the load to some deeper ground level where the passive resistance will equal the load on the base of the conoid. Also the plain pile is suitable for those cases where a hard layer is encountered and can not be penetrated, in which case the pile will act as a column. There are numerous gradations between these two extreme conditions where special types of piles may have an advantage. The pedestal pile is one of these special types. It is made of concrete and has a large ball-like end that provides a much greater bearing area at the bottom than that of the plain pile. The ball is formed by a ramming process at the time the concrete is deposited. This type is especially adapted for the condition where the bottom of the pile rests on or slightly penetrates a thin hard stratum. What effect the ball has on the "conoid of pressure" developed by the skin friction on the stem of the pile has yet to be determined.

Screw piles and disk piles are used for locations where the penetration is limited. Disk piles are sunk by means of a water jet, and hence can only be employed in soils that can be jetted. Screw piles have a large screw blade on the end, and are screwed into place by turning the shank with a capstan. A water jet facilitates the operation.

Lagged piles have been found to develop a greater bearing capacity in soft soils than plain piles of similar dimensions and penetration. The lagging usually consists of four long timbers bolted to opposite sides of the plain pile and extended the length of the desired penetration. The increased surface affords more skin friction and results in a material gain in bearing capacity. For soft soils care should be taken not to space the piles too close together, in order to prevent the "conoids of pressure" from overlapping.

The concrete pile of late has gained rapidly in favor, and is now being used extensively for shallow foundations where the water level is not high enough to keep the piling continually submerged. There are many types on the market, and most of them are patented. These types belong to two general classes, viz., the pre-moulded pile and the pile made in place. The pre-moulded piles have some form of steel reinforcement placed in their interior at the time of fabrication, which gives them strength for handling and driving as well as for carrying loads. These piles should be left in the yard to cure for thirty or forty days before driving. A water jet should be employed in conjunction with the hammer when sinking such piles; and a cap or cushion of rope should be placed on top so as to distribute the pressure and relieve the shock.

To avoid the loss of time involved by the necessity for curing all premoulded concrete piles, several devices have been developed for casting each pile in place. This requires the making of a hole in the ground by a boring tool or by driving a "former" the required depth and then pulling it out, after which concrete is placed in the hole. Some methods provide for leaving in the hole a sheet steel casing and filling it with concrete, and, if desired, inserting reinforcing steel. Other methods involve the drawing of the casing as the concrete is deposited. Which of these two types should be adopted for any case will depend upon the nature of the soil to be penetrated. With soft material or quicksand having a tendency to flow, the sides of the hole are liable to cave in and the material thereof to get mixed with the fresh concrete during its placing, thereby weakening or even entirely ruining the pile. Also the driving of the adjacent "formers" will cause a bulging of the soil so that the holes already filled with fresh concrete will be affected, and there will result a diminished sectional area of the previously poured concrete. A good illustration of this defect is given in *Engineering News* of October 12, 1912. Where a shell is left in the hole these difficulties are effectually overcome.

Too much care cannot be taken to ensure that the poured piles are of uniform section and symmetrical about their vertical axes, and that the concrete does not become segregated during the operation of filling the hole nor mixed with earth. Lack of symmetry produces bending stresses that have a weakening effect, the proportionate amount of which seems almost unbelievable to anyone unaccustomed to the making of strength computations. No single type of concrete pile will fit all locations. Borings should be made before selecting the type for any place, as a knowledge of the materials to be penetrated is essential if one is to avoid utilizing snap judgment in his professional work.

The matter of grouping and spacing piles is important. For bearing purposes, piles should be grouped symmetrically about two rectangular axes. The minimum spacing should be about three feet, unless the piles rest on a hard stratum and are acting as columns.

For sheet piling the Wakefield type is the best of all the wooden kinds. It is built up of three similar planks nailed or bolted together in such a way that the middle plank forms a groove on one edge and a tongue on the other. While single planks placed in single, double, and triple rows have been used, they are not as effective as those of the Wakefield type. However, the interlocking steel sheet-pile is rapidly displacing the wooden patterns for temporary purposes. Some of these steel piles are made up of channels, angles, or zee bars riveted together, while others are rolled in one piece. When stiffness is required, the built-up section has the larger radius of gyration and should, therefore, be employed. The handbooks issued by the steel manufacturers show the various types of sheet piles, and give their properties so that the engineer can readily make his selection for special conditions. Their superior strength, their facility in penetrating hard strata and in cutting through logs, their small leakage, and their ability to stand pulling and redriving make steel piles suitable for operations and conditions that wooden piles cannot meet.

The durability of piling is a feature which must be considered in all permanent construction. In the case of wooden piles continuously saturated in fresh water no decay occurs. Piles have been found in a good state of preservation after being submerged from six hundred to one thousand years. However, if the water level fluctuates and leaves any portion of the pile exposed to the air at any time, decay will eventually set in, unless the timber has been thoroughly treated with some preservative; and it is possible that any such treatment will merely put off the evil day of ultimate destruction. It seems, though, to be the general opinion among engineers that piles of sound timber thoroughly impregnated with creosote will last indefinitely unless placed in sea water. In the latter where the *Limnoria* and teredo abound, timber piles without some protective coating should never be used. Creosoting will delay the attack of these marine borers for a number of years. The life of any treated pile depends a great deal upon the condition of its surface after driving, as mechanical injuries and checks favor the entrance of borers. For protecting piles already in place many mechanical devices have been tried with more or less success. The reinforced-concrete coating is probably the most satisfactory if it be extended sufficiently into the mud. This result is difficult to accomplish on a pile in place, as the concrete mixes with the mud so that it eventually breaks off, leaving a portion of the wood exposed to attacks of the borers. To obviate this difficulty, vitrified clay tiles have sometimes been adopted; and the cement gun has been employed for coating the timber above the low water line. In the latter process a piece of ordinary poultry wire netting is wrapped around the pile and fastened securely thereto; and then a 1½″ or 2″ layer of cement mortar is placed around the timber by means of the gun. The portion of the pile below the low water line, however, cannot be reached by this method.

For new work, a combination pile, consisting of the ordinary round timber with a shell of reinforced concrete enclosing it, is well adapted. The concrete shell, extending well below the mud line, affords the desired protection against the *teredo* and *Limnoria*. The timber core projects several feet above the top of the concrete casing so that a hammer can be used for driving, as in the case of ordinary timber piles, without injuring the concrete.

Concrete piles are equally durable in dry or wet soils, and, of course, are not at all affected by any marine borers. If properly made and not damaged in driving or in placing, they will last indefinitely. They can be made uniformly straight and symmetrical to a much greater extent than timber piles can be procured.

The question of durability seldom arises with the use of steel sheet piles, as they are generally employed merely for temporary purposes. They will ordinarily stand pulling out and re-driving many times; and, if rust is not allowed to accumulate on them, there is a certain amount of salvage obtainable in some types that can be employed for structural purposes after their temporary service as sheet piling is finished.

Various methods have been tried for driving piles. For short ones, especially sheet piling, a maul is sometimes used where small penetrations only are desired. A more effective means is a set of light leads and a hammer made of a 12″ × 12″ timber heavy enough for four men to operate by a rope running over a pulley. If a derrick is employed on the job, it is a very good plan to work the hammer at the end of the boom through a very light pair of leads. This will be more economical than using manpower. In nearly all bridgework a derrick is erected close to the excavation to remove earth from the pit and to place material for the structure. This can readily be used also for driving the piles.

For large piles a drop-hammer operated by a steam engine is generally employed. This hammer, which often weighs several thousand pounds, slides up and down a pair of guides known as leads. These are braced in the rear and on the sides by suitable framework, both the leads and the bracing being attached to horizontal sills at the bottom. This framework can be shifted along on the ground by crow-bars or it can be placed on rollers so as to permit of its being moved into position more readily. It may also be put on a flat car and provided with a turntable so that it can be swung sideways and thus reach positions outside of the track, in which case it is known as a track driver. When the driver cannot be placed close to the ground, it is frequently necessary to provide extensions to the regular leads so that the hammer can follow the pile down below the grade of the track. For driving batter piles, the leads are arranged to swing about a pivot like a pendulum, so as to give an inclination to the pile when it enters the ground and during driving.

In place of the drop-hammer, a steam hammer is frequently mounted in the leads. This apparatus delivers its blows in rapid succession, so that the acquired momentum of the pile is not lost between blows and the soil has no chance to close tightly around it. For quicksand and other sandy soils, this is a desirable feature. Another advantage possessed by the steam hammer is that it does not broom the head of the pile as does the drop-hammer.

Another method of driving is the placing of an explosive on top of the pile. For instance, a charge of dynamite is laid on a thick iron cap resting on the pile-head and is exploded by electricity. The explosion produces a blow on the pile similar to that from a hammer. This method has not come into general use, however, although the scheme is an old one.

Another and effective means of sinking piles is by using two powerful water jets. This method is especially applicable to sandy and gravelly soils. Whenever possible, concrete piles should be driven with water jet, using the hammer only for light tapping when the pile refuses to move. When the jets are fixed onto the pile, they should be placed on opposite sides in order to preserve a balance of pressures, thus tending to make the pile sink true to position. Inexperienced contractors quite often attempt to save expense by employing a single jet attached to one side of the pile, but the result is invariably unsatisfactory. If the jetting pipe were located on the axis of the pile, or even if the nozzle could be placed axially at its point, a single jet would serve the purpose effectively; but these *desiderata* are generally too difficult of attainment to warrant the attempt to secure them, except, perhaps, in the case of a reinforced-concrete pile, which could be cast around a tin tube. In the case of certain clayey soils, it has proved effective to work the jet down first, making a hole, then to withdraw it and sink the pile into place with light blows. When the jet is free from the pile it can be worked around on all sides, thereby reducing the skin friction. It is then necessary, though, to keep the jet moving continuously up and down and sideways; for, otherwise, the loose material will pack around it, causing the pipe to bind and making its removal difficult. Generally the withdrawal should occur before the pile has quite reached its desired penetration, so that the hammer can be used to tap it to final position.

In the harder kinds of clay soils an effective expedient is to prepare a ten-inch pipe about ten feet long with a slot about four feet long cut in one side, and a round timber framed and bolted into one end. This is then driven into the ground a short distance and pulled out, bringing up a chunk of clay inside the pipe. By means of the slot in the side the content is readily removed from the pipe, and the operation of driving, pulling, and cleaning out is repeated until the layer of clay is penetrated or until a sufficient depth has been reached. The concrete pile should be placed in the hole thus made, and then by using a water jet, and by churning and tapping the pile with the hammer, the pile can be driven to the required elevation. This expedient was used on the author's bridge over the Missouri River on the line of the Great Northern R. R. at Mondak, Mont. There a hard clay, known locally as buckshot, was encountered in one of the abutments, and every other device that was tried for driving through it failed.

On the Intercity Viaduct connecting the two Kansas Citys, a single water jet was used in sinking the concrete piles, but in this case it was detached and was worked around them. A two-ton hammer rested on top of the pile, and a wire cable was attached to the head thereof under the said hammer. Then the pile and hammer were raised with the cable and churned up and down until the former reached its final position. In some of the foundations the piles were driven from three to five feet below the ground surface, the pits being excavated later by another contractor. When the driving extended below the surface, a tripping apparatus was attached to the cable around the pile, so that no trouble was experienced in removing it when desired. For driving long piles in sand or gravel no method has been found to compare with the water-jet process when properly handled. When water jets are not used, the steam hammer is more effective than the drop-hammer in sandy and gravelly soils.

The driving of piles by the water jet is quite an old process; but its application to long piles was initiated by the author and his contractor friend, the late C. E. H. Campbell, Esq., Mem. Am. Soc. C. E., on the temporary piers of the East Omaha Bridge in 1893. The contracts for the building of that structure, for business reasons connected with real estate, had to be let privately and without competition; and the task of so letting them devolved upon the author, who naturally selected companies known to him as capable, honest, efficient, and trustworthy. Up to that time no engineer had ever dared to figure on putting in wooden piers for any bridge over the Missouri River; and, consequently, the author's so doing (of which, by the way, he was by no means the instigator) was somewhat in the nature of an experiment. His design involved the driving of a large number of cypress piles 80 feet long some 50 feet or more into the sand. When Mr. Campbell was confronted with the problem he remarked: "I don't know of any piles that have been driven by jetting more than twenty or twenty-five feet, and it may be impossible to sink them the depth that you require." To this the author countered: "But are you not willing to try?"—and being an engineer of courage, he was. The plans for the apparatus were made by Mr. Campbell under the author's direction, using two 2½-inch pipes per pile with reduced orifices, some 350 gallons of water per minute, and a pressure of 100 or more pounds per square inch. The result of the experiment was eminently satisfactory, for no trouble whatsoever was experienced in the sinking, the time required for actual driving averaging only five or six minutes per pile. As a matter of experiment one pile was sunk 60 feet, and no special difficulty was encountered because of the extra depth. Ten years later when the shore piers of the permanent bridge at the same crossing were being built by another contractor, it became necessary to drive some 80-foot piles their full length into the sand at the bottom of an open crib. The contractor said he could not do the job, but under the direct superintendence of the author the feat was accomplished without any serious trouble, the main difficulty encountered being to hold the pile down after driving. This was effected by leaving the hammer on the pile while the pipes were being withdrawn.

Long piles were put down by water jets in two of the author's bridges over the Fraser River at New Westminster, British Columbia. In the larger one the contractors were willing to accept the engineer's advice, and although at first they encountered what they deemed serious difficulty on account of having to drive through gravel, they quickly overcame it by doubling the pumping capacity, after which the driving proceeded smoothly until completion. But on the smaller bridge (Lulu Island) the contractor was unwilling to take advice, and insisted on putting in a totally inadequate plant, with the result that he could not get the piles down to the required depth until after he had enlarged his plant and his pumping capacity.

In the Pacific Highway Bridge over the Columbia River at Portland, Ore., now under construction, as unusually long piles have to be driven through difficult material, the author's firm specified two jets per pile, each discharging 900 gallons of water per minute. It had been stated by high engineering authority that it is impossible to drive such piles for the piers of that crossing, hence the unusual precaution adopted. Up to the time of writing the contractors had not started the pile driving; hence it is impracticable to state here the result of the firm's attempt to "accomplish the impossible." With rose-jets, a pressure of 175 pounds per square inch, and a flow of 1,800 gallons per minute, it will be an unusually well compacted gravel that will prevent absolutely the penetration of the piles—*nous verrons.**

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* *Post Scriptum.* It is now several months since the above was written; and since then most of the piles for the piers—in fact, all of the very long ones—have been driven, and as yet no special difficulty has been experienced in putting them down.

As a guide to those substructure contractors who are willing to profit by the experience of others, the author offers the following data concerning the driving of piles by water jets:

Where the penetration is fifty feet or over, or where the material to be penetrated is unusually hard, there should be provided a pump of the following dimensions:

Diameter of steam cylinder, 16 inches.

Diameter of pump pistons, 8 inches.

Length of stroke, 12 inches.

Revolutions per minute, 50 to 75.

Gallons per minute, 500 to 750.

Suction, 8 inches.

Discharge, 6 inches.

Diameter of jet pipes, 2½ inches.

Diameter of hose, 2½ inches.

Steam pressure, 120 to 150 pounds per square inch.

Boilers with more than enough capacity to supply the pump or pumps under the most unfavorable conditions possible.

Jet pipes of double strength, and the strongest procurable hose.

The nozzle diameter for plain jets should be one and a half inches; but rose-jets also should be provided, as they make a larger hole than do the plain jets, and are exceedingly useful when coarse gravel is encountered. A 50-foot section of hose on each jet will generally suffice, provided that a stand-pipe for each jet is extended half way up the leads of the pile driver.

The difficulties ordinarily encountered in pile driving are due to obstructions, such as boulders or sunken logs, in the path of the pile. A steam hammer is effective sometimes in cutting through logs, while a water-jet will tend to displace boulders. Quicksand makes hard driving unless a water-jet is employed. No pile driver outfit is complete without a water-jet equipment. A layer of cemented gravel is often encountered near the bed in streams; and it must be pierced in order to obtain the desired penetration. If its point be shod with iron, the pile may often be driven through such a layer; but sometimes it is necessary first to drive through it a bar of steel, such as a piece of shafting or a railroad rail, in order to break the crust, after which the pile will penetrate.

One of the most serious troubles to guard against is that of overdriving the pile. The symptoms of overdriving are small penetration per blow, springing and vibration of the pile under the shock, brooming of its head, and a sudden change in the amount of penetration per blow. Penetrations smaller than a half inch should be viewed with suspicion. A number of cases are now on record (see Vol. 10, Part 1, page 572 *et seq.* of the *Proceedings* of the American Railway Engineering Association) where piles were driven without apparent injury, but when exposed by subsequent excavation were found to be shattered and badly broomed at the point, so that the effective bearing was much reduced. More damage can be done by overdriving than by underdriving. A knowledge of soil conditions gained from borings will materially aid the engineer in passing judgment on pile driving.

To estimate the cost of driving piles for any particular job, the engineer should consult cost data obtained from other somewhat similar jobs, consider thoroughly the conditions which are different, and adjust his estimate to correspond. A great deal of cost data is now published in convenient form; and the engineer should consult these records, if his own do not cover satisfactorily the case in hand. The principal factors that will affect the cost of driving are the kind of material to be penetrated, the amount of penetration, the type and material of the piles to be used, the method of driving, the cost of labor, the practicability of using water-jets, the convenience of the location of the piles to the driver, and the traffic restrictions or other possible interferences.

CHAPTER XLIII

PIERS, PEDESTALS, ABUTMENTS, RETAINING WALLS, AND CULVERTS

This chapter treats mainly of the *shafts* of piers, pedestals, abutments, and retaining walls, because the portions of those constructions below water have already been discussed fully in Chapters XXXVIII to XLI inclusive. However, specifications for piers are presented herein so as to bring together in one place the necessary instructions for determining the general features and the details of design of the various types employed in a bridge engineer's practice.

Pier shafts may be divided into the following classes in respect to the materials of which they are built:

1. Stone masonry.
2. Brick masonry.
3. Unprotected concrete, either plain or reinforced.
4. Concrete protected by a belt of stone masonry or steel plates.
5. Oblong steel shells filled with concrete.
6. Cylinders filled with concrete and braced with steel or with steel and concrete.
7. Braced steel towers.
8. Timber towers or cribbing.

In respect to which of these eight kinds of piers it is best to adopt for any particular crossing, the engineer must use his judgment, which, however, may be aided by the following remarks that are based mainly upon the author's personal experience.

Piers of stone masonry for many years were used almost exclusively for important bridges; but today they are seldom employed, as concrete piers are much cheaper and just as good, if not better. It is only occasionally that one now hears of a stone masonry pier being called for in America, although one prominent engineer in this country still exhibits a preference for the more expensive material and appears to be able to persuade his clients to pay for the doubtful luxury. First-class stone is a *sine qua non* for stone masonry piers, because poor stone would not be either as strong or as durable as ordinarily good concrete.

The proper way to proportion a masonry pier is to determine the least size under coping to support either the pedestals themselves or the pedestal-blocks, as the case may be, leaving a small margin on the exterior and ample room between pedestals or pedestal-blocks to allow for variation in erection, then batter the pier all around not less than one-half inch to the foot, or as much more as investigation shows to be necessary.

The coping should project all around about six inches, the amount depending upon the magnitude of the pier and the thickness of the coping course, which should be from eighteen to twenty-four inches.

The batter for the sides is to be determined in the following manner:

First compute for both the loaded and the empty structure the greatest longitudinal components of the total wind-pressure that can come upon the pier from the two spans which rest thereon, upon the assumption that the friction at the roller ends of the spans is zero. The longitudinal thrust is to be taken as seventy (70) per cent of the total transverse wind load. Find also the greatest traction thrusts from braked trains on the assumptions that, first, the greatest live load is on the structure, and second, that the least live load, or one thousand pounds per lineal foot, is on the same. Now find the values of the following combinations:

1. Thrust from wind load on empty bridge.
2. Thrust from heaviest braked train.
3. Thrust from wind load with lightest live load on the spans.
4. Combined thrust from lightest possible braked train, and a wind-pressure on train and structure equal to one-half of that specified.

Next determine by judgment the proper batter, and lay off the pier to scale. Then divide it by horizontal planes from four to six feet apart, and compute the weights of all the portions of the masonry between these planes, making a proper reduction for weight of water in the cases of those parts which would be submerged by an average stage of river.

Next compute the wind-pressure on each vertical division of the pier, down to the assumed stage of water, in a direction parallel to the spans, using the same intensity and direction for the wind-pressure as were adopted in finding the longitudinal thrust from wind-pressure on the bridge.

Next find graphically for all four cases the curves of pressure from the vertical and horizontal loads at top of pier, combined with the weights of the various divisions of the latter and the wind-pressures thereon, and see that none of the said curves at any plane of division pass outside of the middle third of the section at the said plane. If any of them do, the batter will have to be increased, or, if all the curves fall much inside of the middle third points, it will have to be decreased; and in either case the graphical computations will have to be made again, and so on until a satisfactory batter is found.

The author is aware of the fact that this method of designing piers is not in general use, and it is quite possible that he is the sole engineer who adopts it; nevertheless he maintains that it is the only proper way to design masonry piers. The single concession which he would be willing to make on the score of economy would be to assume that a certain small portion of the thrust on a span is taken up at the roller end. But if the rollers are in good working order the amount of thrust that they will resist is very small indeed—so small, in fact, that it is best to neglect it entirely.

The ordinary method of proportioning piers is to make them as small as possible under coping and batter them all around, or at least on the sides, one-half inch to the foot. In some cases this will suffice, but in others it will not. One of the largest bridges in the United States has piers built with such insufficient batter that it is evident at a glance, to even an untrained eye, that something is wrong. One of these piers is cracked from top to bottom, owing to false economy in the design, but not because of its failure to figure properly for the curve of pressure.

An inherent sense of fitness in the mind of the designer will generally tell him, when he looks at a scale-drawing of the superstructure and piers of a bridge, whether the latter are properly proportioned. In the case of the Red Rock cantilever bridge over the Colorado River the piers were first laid out fourteen feet wide under coping, with a batter of half an inch to the foot, and the drawings were submitted to the author for his criticism. He immediately pronounced the piers to be proportioned incorrectly, simply because of their appearance. Their designing was then turned over to him, and he found by trial that a batter of one and a quarter inches to the foot was necessary. This batter gave a satisfactory appearance to the entire layout.

In nearly every case the length of the piers up and down stream, determined by the minimum size under coping and the proper side-batter for thrust, will provide sufficient strength and stability to resist both current and wind-pressure. A thorough investigation of resistance to overturning of piers down-stream is given in Baker's "Treatise on Masonry Construction." In it he proves that any pier which is large enough under coping, and which has ordinary batter, will resist properly the overturning tendency of the worst possible combination of loads from wind, current, and floating ice. Nevertheless, in long-span, single-track bridges with very tall piers, crossing swift streams that carry thick ice, and where the structure is exposed to high winds, it is advisable, as a matter of precaution, to test the piers for down-stream overturning according to Prof. Baker's method. Should the length of pier parallel to the stream be found insufficient, the neatest way to obtain the requisite stability is to put in a cocked-hat just above the elevation of extreme high water.

When a masonry pier rests on bed-rock, the latter should be leveled or stepped off, and there should be placed a layer of rich concrete between the rock and the masonry.

Brick piers are not common in America, probably because, until lately, it has been difficult to obtain proper brick. In the author's opinion, piers built exclusively of hard-burned clinker brick and mortar of the very best quality of Portland cement, mixed in the proportion of one part of cement to two parts of sand, and having thin joints perfectly filled, are better than the average masonry pier, for the reason that the bricks will never disintegrate, while the average stone used for bridge-piers will.

Unprotected concrete piers are satisfactory wherever there is no ice of any account carried by the stream. In order to obtain a smooth, neat finish on the exterior surface of concrete, two methods are employed. The more common one is to spade back the stones from the form by the use of stone-forks. This sometimes produces a fair finish, but too often it does not. When the surface is too rough for appearance, it can be smoothed soon after the forms are removed by rubbing it down with corundum bricks, applied generally by hand but preferably by power; or it can be covered with grouting put on by a pneumatic gun. How durable the coating thus given will prove will depend greatly upon how rough, clean, and wet the surface covered was made and how thoroughly the coating was applied. Troweling on a skin-coat of mortar to even-up the irregularities of surface of a pier is a most unsatisfactory way to improve the appearance of the construction; for the said skin-coat sooner or later peels off and leaves the surface even more unsightly than it would have been had no attempt at all been made to improve it. The other method of securing a smooth finish is described in full detail in the specifications of Chapter LXXIX. It consists in carrying up simultaneously with the concrete an exterior shell of an inch and a half of mortar, keeping the latter separated temporarily from the concrete by small, thin, steel plates that are removed and used over and over again as the work proceeds. This method is a little more expensive and troublesome than the spading-back method; but it is always sure of giving satisfactory results. Contractors dodge its employment all they can—to the detriment of the æsthetics of their work.

Various types of shafts are employed for concrete piers, as they can be formed very readily to suit any condition. The ordinary solid pier with rounded ends is the type most commonly used, because it is the most substantial—besides having the best appearance. However, for either high or long piers, especially those supporting heavy spans and thus requiring widths of considerable amount, this type becomes very expensive on account of the large mass of concrete in the centre that is of little value so far as supporting the loads is concerned. Some of this extra concrete can be easily dispensed with in one of three ways. In the cheapest construction the shaft can be built in two separate cylinders or square columns, each supporting one shoe from the adjacent spans and being battered the same as ordinary piers so as to give them greater strength. Such piers should be employed only for cheap highway bridges over unimportant rivers where there is no danger from ice or drift. A more satisfactory construction connects the two cylinders or columns with a substantial diaphragm, usually of reinforced concrete, forming what is sometimes termed a dumb-bell pier. The two shafts and the diaphragm are connected with a full-width coping, the central portion being reinforced as a matter of precaution, even if it receives no regular structural load. Except for the question of æsthetics, this type of pier can be made as satisfactory in every respect as the solid pier; and it is nearly always considerably cheaper. However, its appearance is objected to by some persons; and where such objections exist, the two shafts can be connected by reinforced side walls, producing in general effect a solid pier. The well thus formed should be covered by a full-width, reinforced coping, extending over both shafts. Moreover, openings should be provided in the side walls at the top and bottom to permit the equalizing of the water level inside and outside of the well as the stage of the river varies, thus preventing any hydrostatic pressure on the walls. Where these walls are very long, they can be supported at intervals by cross walls. This type of pier is not so economical either in materials or in construction as the dumb-bell pier. The details for these various types of piers adopted in the author's practice are fully covered in the specifications at the end of this chapter.

Pivot piers for draw-spans are more and more being constructed of the cellular form as just described. As a rule, they consist of an outer ring, supporting the rollers or the balancing wheels, connected by radial diaphragms to a central core that carries the pivot or centre casting. A large economy is often thus effected.

The copings of concrete piers are sometimes made of stone; but more frequently they are finished off either with rich concrete of small broken stone, or with granitoid mixed in the proportion of one part of Portland cement, two parts of fine granite screenings, and three parts of finely crushed granite.

The method of designing and proportioning concrete shafts is exactly the same as that explained for the designing and proportioning of stone masonry shafts, except that the curve of pressures need not be kept within the middle third of the section, provided that reinforcing rods be employed to take up the tension caused by the bending moments.

As before mentioned, where concrete piers are subjected to abrasion from heavy ice floes, they must be protected by means of a belting course of hard-stone masonry or steel plates for at least the vertical range of the ice. Usually the latter of the two materials is adopted, because it is nearly always more economical as well as more efficient. It will often be sufficient to protect only the up-stream face and a short length of the adjacent sides; but sometimes it will be found necessary to plate the sides for their full length, in which case a complete belting course will be advisable. Where such is the case, especially in piers of moderate height, a steel shell covering the entire shaft may prove economical, for then it can be made to serve as a form, thus eliminating all timber forms except that for the coping. Ice breaks are frequently needed where the ice forms in excessive thicknesses or where the river is subject to destructive ice-jams; and an engineer should always ascertain the probable needs of the case in hand, particularly as the omission of this means of breaking up the ice may prove disastrous to the structure.

When the footing course or the bottom of a pier shaft rests on a crib or caisson, there should be a space of a foot or two all around between the exterior of the shaft and the inner walls of the cofferdam of the said crib, in order to provide for a possible error of final position of the pier base, this offset being a little greater for concrete piers than for those constructed of other materials, so as to permit the placing of the timber forms.

Steel shells filled with concrete make very satisfactory piers, provided they be not used in salt or brackish water, which would rust them out in a short time. Such piers can be built in the usual form of masonry piers with rounded ends all the way up; or, in case of highway structures, when they pass much above high water, they may run off into two cylinders with bracing between. Butt-splices are preferable to lap-joints for the steelwork. This style of pier used to be a favorite one of the author's, for the reason that it is both sightly and inexpensive. When taken to task for using it, as often happened, he used to reply, "Good concrete protected with steel is better than poor masonry." In respect to the thickness of steel to use, the author's practice is to adopt half an inch of metal below the ordinary stage of water and three-eighths of an inch above, although for cheap bridges he occasionally shades these thicknesses one-sixteenth of an inch. For the coping of such piers stone may be employed; but it is preferable to put on a moulding of sheet metal, as this is more in keeping with the rest of the pier. This style of coping has been criticised on the plea that it is false, and that it has no direct function; nevertheless, the author considers it eminently proper to use it, and that its function is simply to beautify the construction by relieving the harsh outlines.

Cylinder piers filled with concrete used to be the most common kind of pier in America, and they are certainly the worst; nevertheless they have their place in good construction, when they are properly designed and built. Their abuse is due mainly to the builders of cheap highway bridges, who think that if the top of the cylinder is simply large enough to hold the pedestals, that is all that is necessary, no matter how high the piers may be, how great may be the scour, or what kind of foundation there may be. If piles are employed as a foundation, they put in all that their small cylinders will hold, and never dream of its being necessary to figure how many tons each pile will have to sustain. Cylinder piers are legitimate construction in places where, under the worst possible conditions in respect to scour, they will have a firm grip in solid material, say not less in depth than twenty per cent of the height of the entire pier. Cylinder piers will not often stand the test of the curves of pressures herein described for masonry piers; but this is not necessary, because they can resist tension on one side in both the metal and the concrete, if the latter be of the correct quality; *i. e.*, the cylinders can act as beams to resist the horizontal thrust of wind and trains in the same way as do many columns of elevated railroads. Nevertheless for railroad bridges the author would advise against the adoption of long cylinders for piers, on account of their inability to resist vibration effectively. In some cases it is economical to adopt a group of four comparatively small cylinders well braced on all four faces; but with this style of foundation it is generally customary to employ braced towers resting on the cylinders. The diameter for a cylinder should depend not only upon the size required at the top, but also upon its height and the character of the foundation. It is sometimes governed also by the total vertical load to be carried, the intensity of which should never exceed the limit set in the specifications given in Chapter LXXVIII. It is economical sometimes to increase the diameter of a cylinder between top and bottom, but in such cases the lower twenty feet should be made plumb so that the cylinder can be sunk with ease and accuracy. This detail was adopted for the Jefferson City Bridge, the variation in diameter being obtained by telescoping some of the lengths and putting in filling-rings. This required a trifle more metal than truly conical piers would necessitate; but the shopwork was much simpler. However, with the present shop facilities conical piers do not present the difficulties they did in the past.

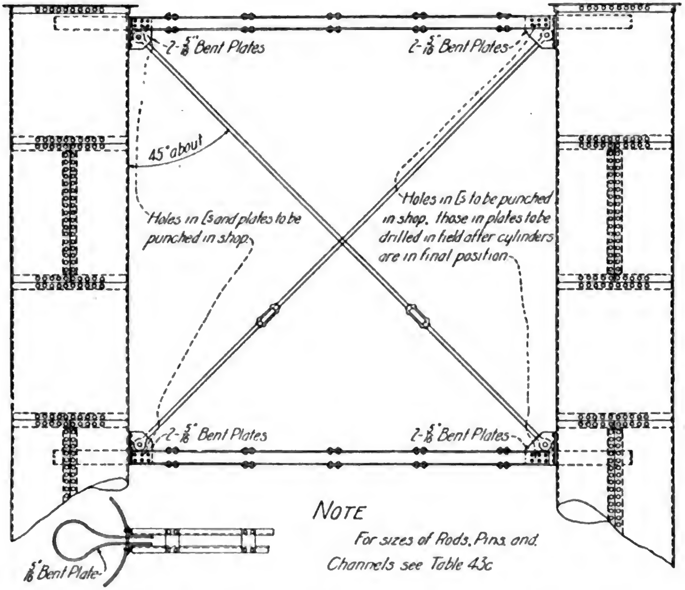
The bracing between the up-stream and the down-stream cylinders of a pier in the river should invariably be of solid webs properly stiffened, extending from high water to near low water, in case there be any drift; but for cylinder piers on shore an open bracing of struts and ties will sometimes suffice. Details of the latter type are shown in Fig. 43*a*. In some of his constructions, notably the Columbia River Bridge at Trail, British Columbia, the author has employed a steel box girder filled with concrete between the two cylinders so as to increase the mass that has to resist the shock of ice and drift-wood. Fig. 43*b* illustrates such bracing.

Portland cement concrete of the very best quality should be used for filling cylinder piers; and the filling should be done with the greatest care and thoroughness. Whenever the concrete has to be placed below water, it should be put in by using a trémie; and the composition of the concrete should be much richer than for concrete laid in the dry.

Whenever a cylinder is sunk to bed-rock, it should be let into the same far enough to prevent all possibility of slipping, and so as to give an even bearing all around the circumference. This is an easy matter when the pneumatic process is employed for sinking, but it is often difficult in the case of open dredging.

Concerning braced steel piers or towers but little need be said, except that they should conform in their design with the specifications given in Chapter LXXVIII. It is advisable, if practicable, to avoid battering more than two faces of a braced pier, on account of the troublesome shopwork that would be involved with a four-face batter; nevertheless it is often necessary to adopt such construction, especially for high piers.

Timber piers are merely a makeshift; hence they do not merit much consideration. They are employed sometimes to support steel bridges until money is available for building masonry piers. It is seldom that timber piers are built in large rivers where the current is rapid and the scour is great. The author was once forced by circumstances into building pile piers under these conditions; and, although they served their purpose excellently, and were in a good state of preservation when removed after ten years of service, he does not recommend their adoption, because of the uncertainty of their ability to resist scour, ice-abrasion, drift-gorges, and fire. The piers referred to were the temporary piers of the East Omaha Bridge over the Missouri River. They were constructed in the winter, mostly on a sand-bar, by driving groups of seventy-foot, red-cypress piles fifty feet into the sand by means of powerful water-jets, then sheathing the sides and the nose of each pier with four-inch oak planks and bracing the piles on the inside. The nose was on an incline, faced with steel plates where the ice could reach it, and forming a cutting edge capped with a heavy railroad rail. Each pier was surrounded with a woven willow mattress, eighteen inches thick, of the most substantial character, sunk and kept in place with rock. These piers received a much more severe test than was anticipated when they were designed, because the channel shifted across the river, so that at times there were thirty-


2-5/16" Bent Plates
2-5/16" Bent Plates
45° about
Holes in [s and plates to be punched in shop
Holes in [s to be punched in shop, those in plates to be drilled in field after cylinders are in final position
2-5/16" Bent Plates
2-5/16" Bent Plates
5/16" Bent Plate

NOTE
For sizes of Rods, Pins and Channels see Table 43c

FIG. 43a. Details of Open Bracing for Steel Cylinder Piers.

five feet of water where there was a dry sand-bar when the bridge was constructed. The mattresses were not injured by the scour, but were simply lowered, the edges going deeper than the portions near the piers. The only ill effect noticeable was the springing down-stream of the tops of two piers, in one case about six inches and in the other about eleven inches. In order to bring the tops of these piers partially back to place and prevent any further deflection, the author employed a detail which proved to be very satisfactory. It consisted in passing one end of a strong iron chain loosely around an up-stream pile and dropping the loop to the bottom, then attaching near the other end of the chain a steel rod with an adjusting device. A number of these chains were used for each pier, the rods passing through heavy timbers on the down-stream end near the top. By screwing up on these adjustments the tops of the two piers were moved back a little. Provision was made for future scour by leaving some spare chain beyond the point at which the rod took hold, so that one chain at a time could be loosened, lowered, and re-tightened.

Pedestals are used where shallow foundations are permissible and where the loads to be supported are moderate. Formerly they were made of stone masonry, but today, in America at least, they are invariably built of concrete. This is generally plain, but it is reinforced when the base is spread rapidly. It is customary, for appearance, to batter pedestals uniformly where they are visible, and for economy to step off those parts of the bases which are always hidden either by the ground or by the water. The top of a pedestal is designed so as to carry properly the greatest possible load that can come upon it; and the bottom is proportioned so as adequately to support the said load plus the weight of the pedestal itself, when due consideration is given to the special character of the foundation. The batter on the faces will generally be about two inches to the foot, although in some extreme cases it might be reduced to one and a half or raised to two and a half inches—or even more, should the total height be small and the foundation soft. When a pedestal rests on rock or other truly hard material, the area of the base should usually be kept small; but when the foundation is soft, either piles should be employed or the base be made unusually large. In most cases the former method is the better, but there are conditions where piles would do more harm than good by breaking up a stiff stratum of clay overlying a much softer one of that or some other material. In such a case it may prove necessary to adopt spread footings properly reinforced, as discussed on pages 857 and 938. Again, it is often necessary to go to a lower elevation with the base when no piles are employed than it is when they are adopted. In designing any pedestal, thought must be given to the possibility of the base ever being exposed either by removal of the surrounding earth or by lowering the water level. If such a contingency is possible, the neat work or shaft should be carried far enough down to prevent the appearance of the structure from being injured by the exposure to view of the unsightly stepping.

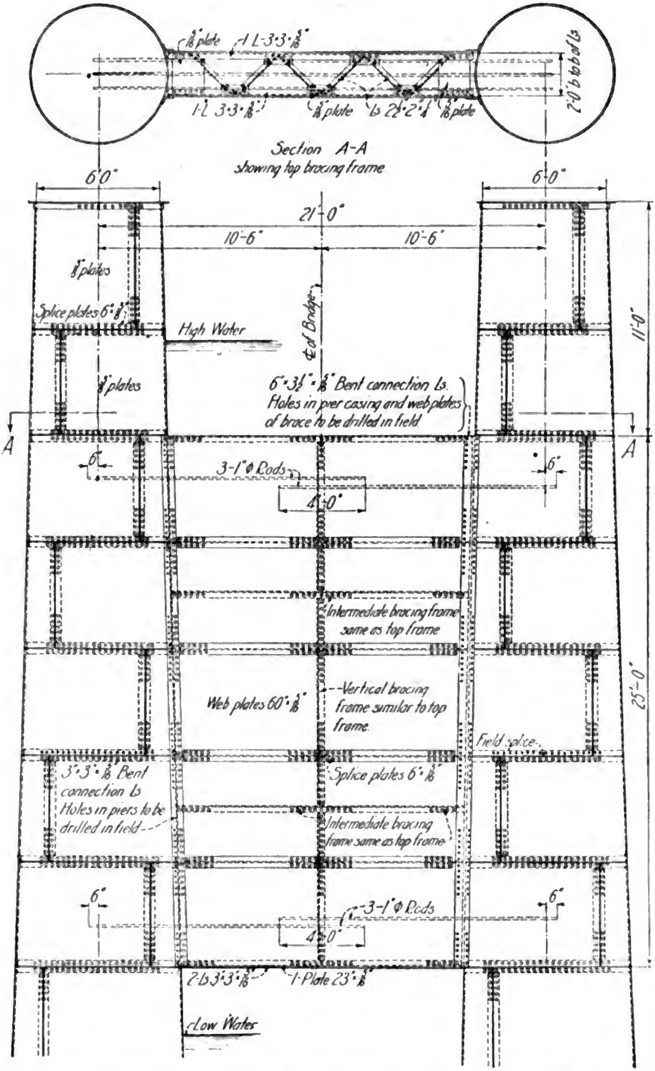


FIG. 43*b*. Details of Solid-web Bracing for Steel Cylinder Piers.

In figuring loads on pedestals the effect of transferred wind load and that of centrifugal load must not be overlooked. Again, especially in elevated railroad work, when solitary braced bents or solitary columns are employed, in addition to the usual loads on the foundation there will be an increased pressure due to bending moments—both longitudinal and transverse. If a pedestal base of dimensions h and b be subjected to a central load P and to a moment M tending to cause rotation in a direction parallel to the dimension h, the unit pressure f at either edge of the base, when $\frac{M}{P}$ (or the eccentricity e) is less than $\frac{h}{6}$, is given by the formula,

$$f = \frac{P}{bh} \pm \frac{6\,M}{bh^2} = \frac{P}{bh}\left(1 \pm \frac{6\,e}{h}\right). \qquad \text{[Eq. 1]}$$

When e is greater than $\frac{h}{6}$, there is a tendency to uplift along one face; and the maximum pressure is

$$f = \frac{2\,P}{3\,b\left(\frac{h}{2} - e\right)}. \qquad \text{[Eq. 2]}$$

When e equals $\frac{h}{6}$, the unit pressure is zero at one edge, and at the other it has the value

$$f = \frac{2\,P}{bh}. \qquad \text{[Eq. 3]}$$

All columns should be anchored to the pedestals in a substantial manner; and if there is any tendency whatsoever to overturning, the connection of the anchor bolts to the steelwork should be such as to develop the full strength of the said bolts. The figuring of anchor bolts is fully discussed in Chapter XVI. In the case of a high, narrow trestle or tower resting on bare rock, there may be a tendency on the part of the designer to cut down the pedestals to a minimum, and this is legitimate so long as the mass employed is sufficient to resist overturning with an ample factor of safety, or in case that the anchor bolts are set a considerable distance into solid rock.

Abutments are used at the shore ends of spans to retain the bank as well as to carry the vertical loads from the span. They should be designed as retaining walls as well as piers. There are four types, viz., straight, wing, U, and T abutments. The size of the top is fixed by the requirements of the span. The size of the base is to be determined by the magnitude and position of the vertical and horizontal loads, and the bearing capacity of the foundation. The necessary conditions to insure stability are treated under the discussion of retaining walls. The centre line of bearing of the shoes should be kept toward the back of the abutment as much as possible. The distribution of pressures on the base should be investigated to see that the unit pressure at the toe does not exceed the allowable bearing value of the foundation; and this is especially true of yielding foundations, where such excess will cause a shifting of the centre of rotation and a rapid decrease in the stability of the abutment. In the case of wing abutments, it will be found that, owing to the load from the span being carried by the head-wall, the latter will have a larger unit bearing than the wing-walls, and hence will tend to settle more than these and thus to produce a crack at their junction. This can be avoided either by making a slip-joint at this place or by putting in sufficient reinforcing steel to make the entire construction act as a unit.

Abutments should be designed so as to secure the following conditions:

The structure must be stable against overturning about any point in the front of the footing or on the face of the wall, and must be safe against crushing at any of these points. It must also be stable against sliding on the foundation or on any horizontal section through the structure. The unit pressure at the toe must be kept within the safe bearing capacity of the foundation. The abutment should protect the embankment against scour and prevent the surface drainage from washing away the earth at the back of the wall; and it should afford an easy and continuous track-connection from the embankment to the span. It should be drained in the same manner as any other form of retaining wall.

Abutments may be made of masonry, of solid concrete, or of the hollow reinforced-concrete type with counterforts. An excellent comparison of the various types with extensive data on quantities and costs is to be found in an article by J. H. Prior, Esq., C. E., which was printed in Vol. 13, pages 1084–1150, of the *Proceedings* of the American Railway Engineering Association. In the Committee Report in Vol. 10, Part 2, of their *Proceedings* a valuable fund of information is to be found concerning the design and details of abutments, and in addition there are given many data regarding abutments actually built.

There is another type of structure known as the buried pier which has some advantages over the customary abutment. In this case the embankment is allowed to spill around a small pier until it is nearly buried, thereby in a large measure equalizing the earth thrusts in front and rear. A short girder span is often used to connect with the main span which rests on a main pier at the bank of the stream. Where there is no danger of scour, this is an economical structure, as it requires no wing walls; and the difference in total cost between a layout with buried piers and one with abutments is often so great as to warrant a considerable expenditure of money for the protection of the embankment against scour by the adoption of rip-rap along its toe at end and sides, carried up to and a little above high-water mark. An advantage possessed by the buried pier layout is that it usually increases the area of the waterway—a matter which sometimes is of considerable importance.

Retaining walls, like abutments, may be of masonry, of solid concrete, or of reinforced concrete either with or without counterforts. They should be designed to act as a unit and to resist the maximum earth pressure that can come against them. To prevent an accumulation of water in the rear of the wall, a drain tile should be laid near the ground-line and covered with broken stone. Weepers should be provided so as to let the water pass off quickly without developing an objectionable hydrostatic pressure against the construction. The factor of safety against overturning about the front toe should be not less than two, and the coefficient necessary to prevent sliding should never be taken greater than 0.4, even when the foundation is rock. The width of base should never be less than four-tenths of the total height, and often this limit should be increased to one-half. Attention must be paid to the distribution of pressures on the base and to the character of foundation, *i. e.*, whether or not it be yielding. With a yielding foundation the point of rotation is shifted back from the face so that the danger of overturning increases rapidly. To study this distribution of pressure on the base as well as to design reinforced-concrete walls it is necessary to approximate the earth pressures acting on the wall.

As a basis for such approximation the author has adopted Rankine's formulæ for conjugate pressures in granular masses devoid of cohesion as giving the limiting pressures to which a wall may be subjected if properly drained. These formulæ have been widely published and are used by many engineers. The American Railway Engineering Association has also adopted them until further investigation develops something more reliable. Friction between the back of the wall and the contiguous earth is neglected so as to secure a greater factor of safety. These formulæ, assuming the wall length to be unity, for a horizontal backing without surcharge are as follows:

Intensity of pressure,

$$p = w\,h \times \frac{1 - \sin \varphi}{1 + \sin \varphi}; \qquad \text{[Eq. 4]}$$

Total pressure (horizontal),

$$P = \frac{w\,h^2}{2} \times \frac{1 - \sin \varphi}{1 + \sin \varphi}; \qquad \text{[Eq. 5]}$$

Overturning moment,

$$M = P \cdot \frac{h}{3} = \frac{w\,h^3}{6} \times \frac{1 - \sin \varphi}{1 + \sin \varphi}; \qquad \text{[Eq. 6]}$$

where w = weight of earth in pounds per cubic foot,
h = height of wall in feet,
and φ = angle of repose of material forming the backing.

For a vertical wall with a surcharge at an angle δ with the horizontal, the total pressure (parallel to surface slope) becomes

$$P = \frac{1}{2} w h^2 \cos \delta \times \frac{\cos \delta - \sqrt{\cos^2 \delta - \cos^2 \varphi}}{\cos \delta + \sqrt{\cos^2 \delta - \cos^2 \varphi}}; \qquad \text{[Eq. 7]}$$

and where δ is equal to φ, this formula reduces to

$$P = \frac{1}{2} w h^2 \cos \varphi. \qquad \text{[Eq. 8]}$$

Attention should be called to the exact agreement between the Rankine formula for total pressure on a vertical face with level back-fill without surcharge and the corresponding Coulomb formula, which is based on the maximum sliding earth wedge, viz.

$$P = \frac{1}{2} w h^2 \tan^2 \left(45° - \frac{\varphi}{2}\right). \qquad \text{[Eq. 9]}$$

As representing average conditions, the author assumes

w = 110 lbs. per cu. ft.,

and φ = angle corresponding to 1½ to 1 slope.

We then have the following formulæ:

For no surcharge,

$$p = 31.4\ h, \qquad \text{[Eq. 10]}$$

and

$$P = 15.7\ h^2. \qquad \text{[Eq. 11]}$$

For a level surcharge of height h',

$$P_s = 15.7\ h\ (h + 2\ h'). \qquad \text{[Eq. 12]}$$

For an inclined surcharge of indefinite extent making an angle with the horizontal equal to a 1½ to 1 slope,

$$P'_s = 45.8\ h^2. \qquad \text{[Eq. 13]}$$

The corresponding values of the moments are

$$M = 5.2\ h^3, \qquad \text{[Eq. 14]}$$

$$M_s = 15.7\ h^2 \left(\frac{h}{3} + h'\right), \qquad \text{[Eq. 15]}$$

and

$$M'_s = 15.3\ h^2. \qquad \text{[Eq. 16]}$$

Between the above limiting conditions of surcharge there will be found various combinations in actual practice, and the designer in such a case will have to choose such an intermediate value as will best fit the conditions. Fig. 43*c* will be found convenient for determining pressures and positions of resultants for the case of no surcharge, that of a level surcharge, or that of an inclined surcharge of indefinite extent on a 1½ to 1 slope. This diagram was prepared on the assumption that w is equal to 100 pounds per cubic foot; and the values for any other unit weight of back-filling can be readily computed by direct proportion, using a slide rule.

Having approximated the earth thrust, it is then combined with the weight of the wall and any earth prism resting on the back thereof; and

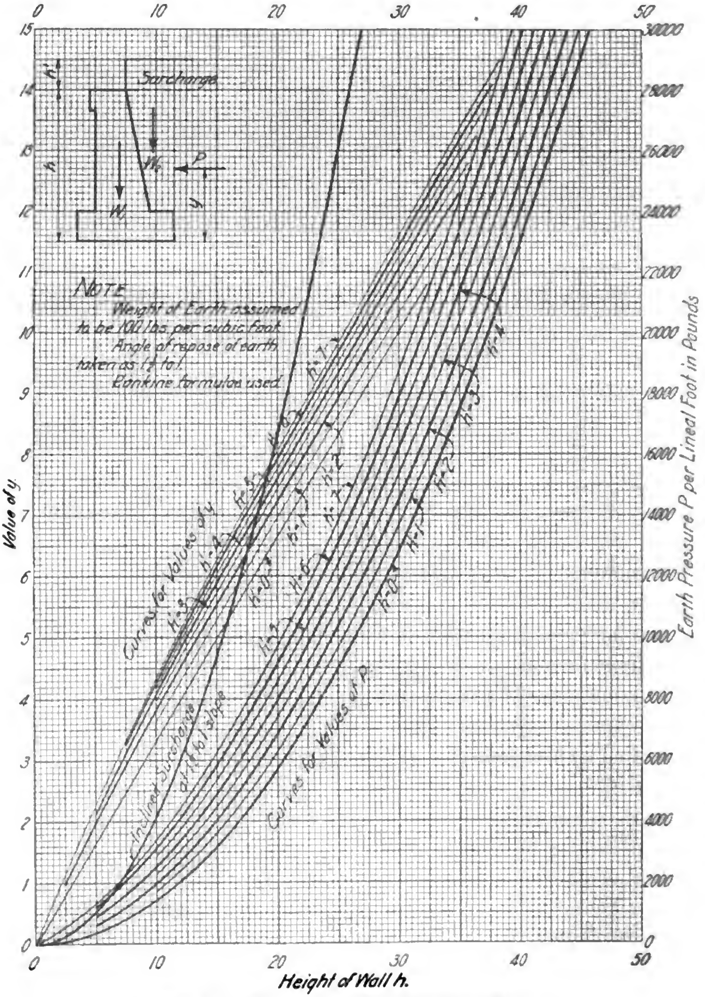


FIG. 43c. Earth Pressures for Retaining Walls.

NOTE.—For inclined surcharge P is parallel to surcharge slope.

the resulting force is found by the usual process. This resultant by its amount, position, and direction determines the upward pressures and their intensities which act along the base of the wall. These intensities are considered to vary as do the ordinates to a straight line. For their determination we proceed as follows:

Let V = vertical component of the resultant,
B = full width of base,
and Q = distance from toe to the force P at base.

Then when Q is equal to or greater than $\frac{B}{3}$, the pressure f at the toe is

$$f = (4\,B - 6\,Q)\,\frac{V}{B^2}; \qquad \text{[Eq. 17]}$$

while at the heel it has the value,

$$f = (6\,Q - 2\,B)\,\frac{V}{B^2}. \qquad \text{[Eq. 18]}$$

When Q is less than $\frac{B}{3}$, the maximum unit pressure is given by the formula,

$$f = \frac{2V}{3Q}. \qquad \text{[Eq. 19]}$$

In no case should the intensity of pressure at any part of a foundation exceed the allowable bearing value for the material considered.

There are two types of reinforced-concrete retaining walls, the cantilever and the counterforted.

The cantilever type is suitable for low walls only—say twenty (20) feet high as a maximum. As shown in Fig. 43*d*, it consists of a vertical face-wall or stem supported by a base-slab, with reinforcement as required. For economy, the stem should be so located that the toe projection is about one-third of the width of the base.

The equations already given in this chapter will suffice for finding the foundation pressures or for computing the width of the base. To determine the sections of the wall, it is then necessary to figure the stresses at the bottom of the face-wall and in the footing-slab at both the front and the rear thereof. The section at the bottom of the face-wall is to be figured for the moment and the shear produced by the horizontal earth pressure and for the direct load of the face-wall itself. Fig. 37*m* will be found useful for this purpose. The unit shear at this place should be kept low for the reasons given later. Evidently reinforcement will be required in the rear face only. The thickness at the top is made the minimum desired—rarely less than twelve (12) inches—and the wall is battered uni-

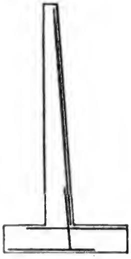


Fig. 43*d*. Cantilever Retaining Wall.

formly from top to bottom at the back. The section of the footing at the front of the face-wall is to be figured for the moment and the shear produced by the upward pressure of the foundation beneath, reduced properly for the weight of the footing itself. The design is to be made in accordance with the notes given on pages 861 and 939 under the headings "The Calculation of Stresses in Wall Footings," and "The Design of Wall Footings." Reinforcement is needed in the bottom only. The section at the back of the face-wall is to be proportioned to resist properly the downward moment and the shear produced by the earth above and the footing slab, reduced by the upward moment and the shear of the foundation pressure beneath. The critical section should be taken at the centre of the face-wall reinforcement rather than at the back of the wall, as this reinforcement furnishes the reaction at this point. It will be found that reinforcement is required here in the top only of the footing. In this instance the critical section for diagonal tension is to be taken at the face-wall reinforcement, rather than at a distance equal to the depth of the footing away from the face, as would be the case were the resultant pressure upward rather than downward.

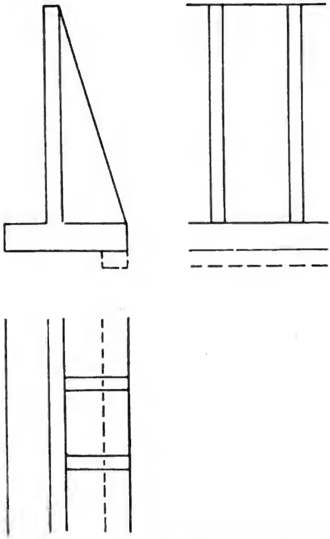


FIG. 43c. Counterforted Retaining Wall.

All sections of the concrete should be made so thick that shear reinforcement is unnecessary. Care must be taken to see that all bars are extended far enough past the critical sections to develop them properly.

In the case of the face-wall reinforcement, this is frequently a difficult matter; and it may require the use of a deeper footing than would otherwise be necessary. Comparatively small bars should be used for this reinforcement, in order that the length of embedment required may be kept down; and they should either end in a hook or else bend toward the front of the footing with a radius equal to fifteen (15) times the diameter of the bar, and then extend along the bottom of the footing to serve as reinforcement for the toe. As the face-wall reinforcement will be spliced a short distance above the top of the footing, this detail does not involve the use of unwieldy bars; and it is probably the best arrangement possible. Longitudinal reinforcement should be employed in both face-wall and footing to serve as both distribution steel and temperature reinforcement. About one-half of one per cent should be used, unless the expansion joints are placed close together, in which case a somewhat smaller amount will suffice. It should be thoroughly wired to the main reinforcement at all intersections. At expansion points there should be a dovetailed joint in the face-wall; and it may be necessary to thicken the said wall at these points on this account. The only weak point in the cantilever type of wall is the section of the face-wall at the top of the footing. Since this is necessarily a construction joint, its shearing strength is likely to be low. A rather low unit shear should be used in designing this section, as has been previously stated; and in the construction care should be taken to see that the surface of the concrete of the footing is well roughened and that it is thoroughly cleaned and wetted before the placing of the concrete in the face-wall is begun.

The counterforted type of retaining wall is preferable to the cantilever form for heights exceeding twenty (20) feet, and is at least as satisfactory for somewhat lower walls. As shown in Fig. 43*e*, it consists essentially of a thin face-wall resting on a footing-slab, and tied back to the footing by vertical counterforts at intervals. The front wall is usually placed some distance back from the front of the footing, a maximum of economy being obtained when the front projection is about one-third of the total width of the base. Occasionally buttresses from the face-wall to the footing-slab are used in front at each counterfort; but this is unusual.

The pressures on the foundations and the width of the base are to be determined as for the gravity or the cantilever type of wall, using the formulæ previously given in this chapter. After these have been figured, it is necessary to design the face-wall, the front toe, the footing-slab back of the face-wall, and the counterforts.

The face-wall is to be designed as a horizontal beam, loaded with a uniform horizontal load, and supported by the counterforts. The load at any point can be figured by means of equations previously given. The wall will be continuous over the counterforts, except at an occasional expansion joint, and even there it will be monolithic with the counterfort; so that the moment at each counterfort and at the centre of each span should be taken as $\frac{p\,l^2}{12}$, where p is the unit pressure at any given elevation, and l is the distance from centre to centre of counterforts. The reinforcement can be arranged as shown in Fig. 43*f*. Bars "1" and "2" are used alternately, Bars "3" being in the same horizontal planes as Bars "1." For low walls, the face-walls can be of constant thickness throughout, the spacing of the reinforcing bars increasing gradually from the bottom to the top; but for high walls, it will be advisable to make the thickness at the bottom greater than that at the top. The minimum thickness of any wall should be twelve (12) inches.

The design of the front toe, for the usual case in which no buttresses are employed, is identical with that of the cantilever type. However, the moment from this toe, instead of being resisted by the face-wall as in that form, is taken up by the base-slab back of the face-wall; so that in this case the reinforcement of the toe must be carried back into the rear portion of the said base-slab, none of it being bent up into the face-wall.

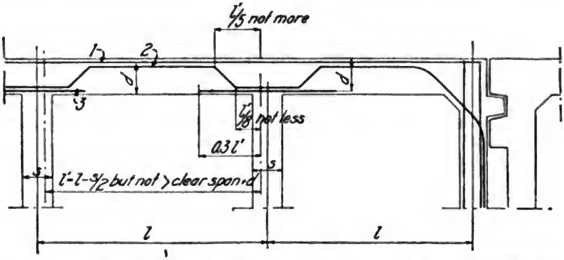


FIG. 43*f*. Reinforcement of Face-wall of Counterforted Retaining Wall.

The design of the base back of the face-wall is somewhat complicated. A panel of this base is evidently a slab supported along three edges by the face-wall and the two counterforts, being continuous over the said counterforts (except at expansion points, where it is continuous with a fairly stiff counterfort) and practically simply supported at the face-wall. In some cases a girder is placed along the back, as indicated by the dotted lines in Fig. 43*e*; and this furnishes a support along the fourth side of the slab, which support, however, is likely to be somewhat flexible as compared with the face-wall and the counterforts. The external loads and moment on this slab are as follows:

First. A uniform downward pressure over the entire area, due to the weight of the slab and the earth above it.

Second. An upward pressure from the foundation, usually varying from about zero at the back to a value approximately equal to the unit pressure from above at the face-wall.

Third. A moment from the front toe.

These three loadings are indicated in the sketch at the top of Fig. 43*g*. The resultant of the first and second loads is a downward load varying uniformly from a maximum at the back to about zero at the face-wall, as shown by the triangle in the centre sketch of Fig. 43*g*. If there is no rear girder, practically the entire amount is carried by the longitudinal reinforcement directly to the counterforts, thus producing stresses in the longitudinal reinforcement and reactions on the counterforts varying from a maximum at the back to about zero at the face-wall. If there is a rear girder, a considerable portion—usually about one-third—of this loading is carried by the transverse reinforcement to the rear girder and the face-wall, and the remainder is taken to the counterforts as before. The loads on the longitudinal reinforcement and the reactions on the counterforts in this case will then vary about as shown in the bottom sketch of Fig. 43*g*. The moment from the front toe tends to bend the rear slab downward, as indicated by the heavy dotted line in the top part of Fig. 43*g*, and therefore puts downward loads on the longitudinal reinforcement, which loads will be roughly proportional to the distances of the dotted line below the full one. A careful study of the deformations of the slab indicates that the deflection below the straight line reduces to practically zero at a distance from the back of the face-wall equal to about six-tenths (0.6) of the spacing of the counterforts. We may, therefore, assume the loads on the longitudinal reinforcement and the reactions on the counterforts from this cause to vary in accordance with the ordinates of a parabola, as shown in the two lower sketches in Fig. 43*g*, the centre of gravity of the parabolic area being located at a distance from the back of the face-wall equal to three-tenths (0.3) of the counterfort spacing. The maximum ordinate to this parabolic area will usually be found to be considerably greater than the maximum pressure due to the first and second loadings, and will, consequently, determine the thickness of the base-slab. Evidently, therefore, the use of the rear girder will not ordinarily permit any material reduction in thickness of the base-slab.

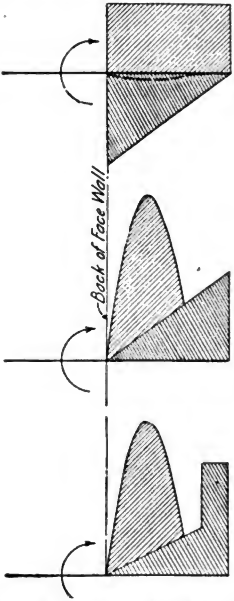


FIG. 43*g*. Loads on Base Slab of Counterforted Retaining Wall.

The sketches in Fig. 43*g* were drawn on the assumptions that the length of the front toe is about one-third of the total width of the base, and that the distance from the face-wall to the back of the base is about equal to the counterfort spacing. For other proportions the load diagrams will change more or less, but the method of analysis still applies; and the centre of gravity of the parabolic area will continue to be about three-tenths (0.3) of the counterfort spacing back of the face-wall. With a small toe projection the intensity of the parabolic loading will decrease, and will no longer determine the required thickness of the base-slab. In that case the use of the rear girder will be economical.

The designing of the longitudinal reinforcement of the base-slab is simple, after the loads are known. As in the case of the face-wall, the moment at each counterfort and in the centre of each span is to be considered as equal to $\frac{p\ l^2}{12}$. Bent-up bars should rarely be employed, it being preferable to use full length ones in the bottom and short ones in the top at each counterfort. The section should be made so thick that shear reinforcement will not be needed.

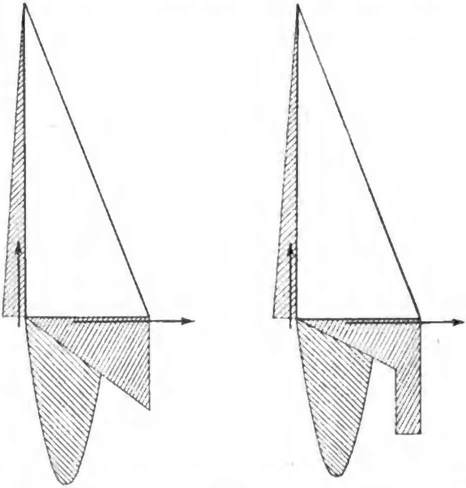


FIG. 43*h*. Loads on Counterfort of Counterforted Retaining Wall.

The counterfort is essentially a triangular beam, loaded along the front edge with the horizontal earth loads from the face-wall, and along the bottom with the reactions from the base-slab. For walls of ordinary proportions in which the rear girder is not used, these loads will be about as indicated in the left-hand diagram in Fig. 43*h*. To balance the horizontal earth load, there will be a horizontal shear of equal amount between the counterfort and the base-slab; and there must also be a shear between the face-wall and the counterfort to balance the load from the slab beneath. If a rear girder is employed, the loads will take the form indicated in the right-hand sketch in Fig. 43*h*. The total vertical load on the counterfort will be somewhat less than in the first case, since a portion of this load is taken directly to the front wall. If in any case the front toe projection is made less than one-third of the width of the base, the parabolic area will be reduced, as has been previously stated.

The usual method of reinforcing a counterfort is to place a sufficient amount of steel along the back edge to carry the moment produced on any horizontal section by the horizontal forces from the face-wall above the section. The stress in these bars is found by dividing the moment by the distance from the centre of the face-wall to the steel, measured normal to the said steel. Enough horizontal steel, well bonded to the face-wall, is then added to take care of the horizontal loads from the said face-wall; and it is carried back and looped around the main reinforcing rods. A sufficient amount of vertical steel is also used to take care of the load from the base-slab. This reinforcement should be well bonded to the latter; and it must be carried up until the stresses can be transferred to the main rods in the back through shear in the concrete.

This style of reinforcement is the best possible for a wall with a short toe projection and a girder along the back of the slab, as shown in Fig. 43*i*; for in this case the larger part of the footing load is delivered directly to the main bars by the rear girder, and the vertical bars have little stress to carry. It also serves well for a wall with a short toe projection and no rear girder, as in Fig. 43*j*; but in this case the main bars should be spread at the back, as the heavy loads from the slab will occur over a considerable width. For a wall with a long front toe, this form of reinforcement is not so well adapted, as the greater portion of the load from the footing-slab is no longer delivered to the counterfort near the back, and there is really no need for very heavy reinforcement at this point. It can still be used, however, with a small loss in economy. The vertical

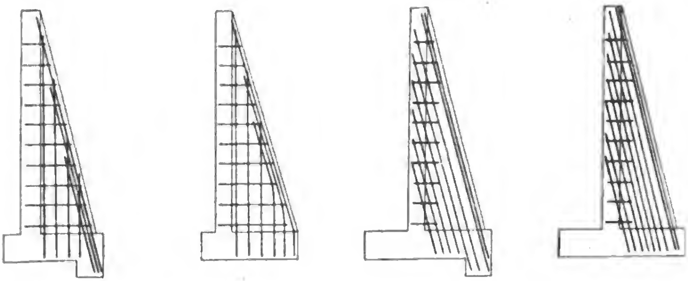


FIG. 43*i*. FIG. 43*j*. FIG. 43*k*. FIG. 43*l*.

Arrangements of Reinforcement for Counterforts of Counterforted Retaining Wall.

reinforcement will have to be quite heavy, especially near the front; and the bars will have to extend up for a considerable distance before their stresses can be delivered to the main reinforcement without overstressing the concrete in shear. It is better adapted to the wall with a rear girder than to one without such a girder. In either form it will be unnecessary to assume that the main bars resist the entire bending moment near the bottom of the counterfort, as the vertical steel will take care of a large proportion of it.

A type of reinforcement that is somewhat better adapted to take up the loads shown in Fig. 43*h* is indicated in Figs. 43*k* and 43*l*. It will be found that the bars, when placed parallel to the back of the counterfort, will be at about the proper inclination to take care of both the tensile and the shearing stresses on its front and bottom edges. A small amount of horizontal reinforcement should be used as indicated, to take care of the possibility that at some point the shearing stress may happen to be low, under which condition the inclined bars would be ill-adapted for carrying the horizontal loads; and in the upper portion it will be best to design the steel along the back on the assumption that true beam-action exists, as this is quite certain to occur. The amount of extra steel required for these two reasons will be small.

At expansion joints a double counterfort must be employed, as shown in Fig. 43*f*. These two counterforts need not be as thick as those used at intermediate points, and they should be keyed together as indicated.

Counterforts should be spaced a distance apart, measuring from centre to centre, equal to $0.66 \sqrt[4]{h^3}$, but in no case less than seven feet, and the thickness of the intermediate counterforts should, preferably, be made about one-twentieth of the height of the wall.

Fig. 56*r* shows the quantities for reinforced walls. The curves of quantities of concrete and metal and those of toe pressures are based on economical sections, as just described. When the supporting power of the foundation is less than that given by the toe pressure curve, the auxiliary curve showing the relation between the ratios of toe pressures and of quantities can be used to determine whether it is more economical to reduce the toe pressure by extending the base or to employ piles, the spacing thereof being about three feet centres. In this connection the increased amount of excavation for the wider base must be taken into consideration.

Fig. 56*s* gives similar information for plain concrete retaining walls.

When piles are employed for retaining walls or abutments, they are not necessarily arranged uniformly over the whole base as is done for piers; but they should be placed where they will resist the loads most advantageously. In the case of retaining walls this means that the majority of the piles will be located near the front toe, although additional piles may be needed near the centre and sometimes at the rear of the base. In abutments the arrangement may vary from that just mentioned to a uniform distribution over the whole base, depending on the location of the centre of pressure. For reinforced-concrete retaining walls and abutments the base-slab proper should be built on top of a plain concrete base from twelve to fifteen inches thick encasing a corresponding length of the piles. The loads on the piles can be determined by means of Equation 26 given on page 300 and developed for rivet groups with bending and direct stresses. The moment and load used in this equation must be taken equal to the total overturning moment and total load divided by the number of rows of piles parallel to the axis of rotation.

For small openings in embankments culverts are used. These may be of iron pipe or reinforced-concrete pipe for waterways only a few square feet in area; but for larger ones a box or an arch culvert is employed. For determining the necessary area of opening to pass the flood water see Chapter XLIX. Pipe culverts should be laid on a slope to increase their discharge capacity. The head of the pipe should be well above the bed of the stream while the foot should be flush with it or a trifle below in order to avoid a sudden drop and scour. A concrete portal is an advantage, as it prevents the water from working around the outside of the pipe and washing out cavities that would endanger the embankment above. Box or arch culverts should be provided with wing walls; and in case the bed of the stream is subject to scour, a concrete pavement should be put in so as to prevent undermining of the head walls. Spandrel walls should be thoroughly anchored to the arch barrel or roof of the culvert; and they should have sufficient stability to resist the thrust from the surcharged earth fill with its superimposed live load. In some cases this thrust may be sufficient to require longitudinal reinforcing in order to resist the tension set up in the arch barrel. Standard railroad culvert plans, with an elaborate set of tables showing dimensions and quantities, are to be found in Vol. 10, Part II, of the *Proceedings* of the American Railway Engineering Association.

In the mixing and placing of concrete for structures of various kinds, the following practical points should be observed in order to secure the best results:

Wet concrete is preferable to dry, and where unskilled labor is employed it is safer.

Tramping or treading is better than tamping.

A mixture of broken stone and gravel is preferable to stone alone.

A mixture of coarse and fine sand is preferable to either alone.

Mud in any form is injurious.

Smooth stone is inferior to rough.

Stone dust is not detrimental, but, on the contrary, is preferable to sand.

Sometimes clay in fine particles up to ten per cent of the total aggregate is not injurious; but the author, as a matter of precaution, prefers to keep it out altogether.

Dry mixtures of broken stone concrete cannot be compacted properly, owing to the arching effect they develop. The best result is obtained by depositing the concrete in layers six to ten inches in thickness.

Prolonged mixing increases materially the strength of concrete.

The best mixture for reinforced concrete in arches and small work is one part of Portland cement, two parts of sand, and four parts of broken stone. For concrete in large masses these proportions should be changed to one, three, and five.

Retempering of mortar is generally injurious, although some experiments seem to indicate the contrary. It is better not to permit it.

For connecting layers of concrete there should be used a wash composed of one part of cement to one part of sand.

In the facing of concrete piers, mortar mixed in the proportion of one part of cement to two parts of sand should be employed.

Plastering rough faces is very unsatisfactory, as the mortar will not adhere properly to the concrete. It has been done successfully at times, but usually it is a failure. When attempting this, the places to be patched should be thoroughly wet, completely saturating the concrete if possible.

In regard to freezing, it has been shown that Portland cement mortars, as a rule, suffer no surface disintegration therefrom, but the strength is generally injured somewhat—sometimes as much as twenty-five per cent, or even more. It does not injure concrete, however, more than five, or at the very most, ten per cent. Freezing after setting does very little harm, but freezing before setting injures the concrete to a certain extent. Alternate freezing and thawing of unset concrete are far worse than freezing and remaining frozen until the final thaw-out. Portland cement concrete, if frozen before the hard set is acquired, will disintegrate when exposed to water and ice. It is better to use a comparatively quick setting cement, if the concrete is liable to be frozen.

Salt added to the water lessens the bad effect of freezing. Some experiments have shown that the addition of salt improves the strength—others that it injures the mortar slightly. The percentage of salt, measured by weight, to be added to the water varies from five to ten.

The use of hot water in mixing mortar and concrete does not tend to improve the quality, especially if the cement is fresh; nevertheless, in very cold weather, it is often a wise precaution to employ it, notwithstanding the fact that the strength is sometimes materially diminished.

Sea water is not good for mixing concrete, hence it is safer not to employ it.

For the results of some important tests on the effect of adopting impure sand in cement mortar, see *Engineering News*, Vol. LIII, page 127.

The washing of sand is generally undesirable, as the finer portions are removed and voids are left.

The best sand is that composed of grains of assorted sizes such that the percentage of voids is reduced to a minimum. Clay, if thoroughly distributed throughout the sand, is beneficial up to twelve per cent adulteration; but clay in lumps is always injurious.

An excessive tamping of the concrete above a joint will add greatly to the value of the bond.

Concrete placed in sea water should not be porous, but should be a dense, rich mixture, in order to prevent infiltration. Good proportions for such concrete are one, two, and three.

SPECIFICATIONS FOR PIERS

Design

Loads

All piers shall be designed so as to be stable against overturning, sliding, or crushing under the following system of forces:

Vertical Loads.

1. Dead and live loads on shoes.
2. Weight of pier reduced for buoyancy.

Vertical Reactions.

1. Allowable bearing on foundation (see Chapter XXXVIII).
2. Skin friction of soil taken at 600 pounds per square foot.

Horizontal Loads (*Parallel to bridge tangent*).

1. Thrust from wind load on empty bridge.
2. Thrust from heaviest braked train.
3. Thrust from wind load with lightest live load on spans.
4. Combined thrust from lightest possible braked train and a wind pressure on train and structure equal to one-half of that specified.
5. Wind pressure on shaft of pier above water line.

Horizontal Loads (*Perpendicular to bridge tangent*).

1. Thrust from wind on empty bridge.
2. Thrust from wind with lightest live load on spans.
3. Thrust from wind on end of pier.
4. Pressure of water against end of pier.
5. Pressure of ice against end of pier.

Horizontal Reactions.

1. Resistance of pier to sliding on base.
2. Passive thrust or resistance of material penetrated.

To determine the pressure of water against the end of a pier the following formulæ may be used:

For square piers,

$$P = 1.24\ A\ V^2; \qquad \text{[Eq. 20]}$$

For circular piers,

$$P = 0.62\ A\ V^2; \qquad \text{[Eq. 21]}$$

For piers five or six times as long as broad, with cutwaters the faces of which make an angle of 30°,

$$P = 0.46\, A\, V^2; \qquad \text{[Eq. 22]}$$

For piers three times as long as broad, with flat ends,

$$P = 1.29\, A\, V^2; \qquad \text{[Eq. 23]}$$

where P = total pressure in pounds on end of pier,
A = area in square feet of vertical projection of pier-end exposed to the current,
and V = velocity of current in feet per second.

To determine the pressure of ice on the end of the pier, assume that the crushing strength of the moving ice is to be overcome. This strength varies from 400 to 800 pounds per square inch.

The passive thrust or resistance to horizontal displacement of the material in which the pier is embedded may be approximated by Rankine's formula,

$$P_e = \frac{w\, h^2}{2} \times \frac{1 + \sin \phi}{1 - \sin \phi}; \qquad \text{[Eq. 24]}$$

where P_e = total resistance,
w = weight of a cubic foot of material,
h = depth of embedment of pier,
and ϕ = angle of repose of material, due allowance being made for its saturated condition.

This formula gives better approximations for granular material, such as sand or gravel, than for other substances.

In case the pier is founded on piles, the heads of which project into the concrete base, the shearing resistance of the piles should be considered as opposing any horizontal motion of the pier, in addition to the resistance of the embedding material.

Unit Stresses

The allowable intensities of working stresses (no impact allowance being included) for the higher-grade timbers, such as Long-leaf Yellow Pine, Douglas Fir, Pacific Coast Cedar, Western Hemlock, and White Oak, shall be as follows:

Tension	1,200	lbs. per square inch
Bending on extreme fibre	1,200	" " " "
Shear with the grain	170	" " " "
Longitudinal shear in beams	110	" " " "
Shear across the grain	1,000	" " " "
Compression with the grain	1,200	" " " "
Compression across grain		
For oak	450	" " " "

For other timbers	250 lbs. per square inch
Compression on columns	
Under 15 diameters	900 " " " "
Over 15 diameters	$1{,}200 - 20\frac{l}{d}$ " " " "

where l = length in inches,
and d = least dimension in inches.

For the lower-grade woods, such as the Soft Pines, Spruce, Tamarack, and Redwood, the following stresses shall be used:

Tension	900 lbs. per square inch
Bending on extreme fibre	900 " " " "
Shear with the grain	100 " " " "
Longitudinal shear in beams	70 " " " "
Shear across the grain	700 " " " "
Compression with the grain	900 " " " "
Compression across the grain	150 " " " "
Compression on columns	
Under 15 diameters	680 " " " "
Over 15 diameters	$900 - 15\frac{l}{d}$ " " " "

where l and d have the same values as before.

In applying the above unit stresses, the actual not the nominal, dimensions of the timber are to be used.

The permissible intensities for the various grades of steel shall be as given in the specifications in Chapter LXXVIII.

The permissible load on a pile shall be taken at from twenty (20) to thirty (30) tons, although in extreme cases where the piles are very long, say from eighty (80) to one hundred (100) feet, and driven into firm material, this load may be increased to forty (40) tons. Where the piles are in a very soft foundation, the load per pile may have to be reduced to fifteen (15) tons or even to ten (10) tons. In any case the judgment of an experienced engineer should decide a matter of so much importance as that of pile-loadings.

When wind loads are combined with direct loads, the above loads per pile shall be properly increased, the excess allowance depending on the relative size, importance, and probability of the two loadings, the ordinary excess for a combination of wind loads with direct loads being thirty (30) per cent of the allowance for direct loads alone.

DETAILS OF DESIGN AND PROPORTIONING OF PARTS

CONCRETE AND MASONRY PIERS

General

The construction of the crib and caisson of a pier depends upon the method of sinking adopted. Typical details are fully illustrated in Chapter XL on the "Open-Dredging Process" and in Chapter XLI on the "Pneumatic Process."

Timber and Sheathing

The timbers in caissons should not be less than 12″ × 12″, except where the construction demands otherwise; and this section is to be preferred to larger sizes.

For cribs and for the wells of deep open-dredging piers down to a depth of 40 feet below the ordinary water level, 10″ × 12″ timbers on edge are to be used; while below this elevation they should be increased to 12″ × 12″.

All caisson and crib timbers should be made of full length, if possible; but when this cannot be done, they must be of the maximum lengths obtainable, and their splices must be so arranged in the different courses that not more than fifty (50) per cent of the timbers are cut in one vertical plane. The splice should, preferably, be made at a bracing point.

The walls and roof of the working chamber of any pneumatic caisson should always be covered with 3″ sheathing, while 2″ stuff will suffice for lining the wells of open-dredging piers. The outside sheathing for both pneumatic and open-dredging piers should be of 2″ material for depths less than thirty (30) feet and of 3″ for depths greater than this, the boards being placed vertically. They should be full length, or else should break joints at least four (4) feet, not over one-half of the sheathing being cut at the same elevation.

All timber and sheathing shall be surfaced on four sides; and it shall be dressed for caulking, where required.

All timber shall be framed in a substantial manner, so as to give the greatest strength possible. Details of such framing are shown in the illustrations given in Chapters XL and XLI.

Size of Caisson and Crib

On account of the difficulty in sinking a pier to exact position, the base should project all around from two (2) to three (3) feet beyond the sides of the shaft. For concrete piers there should be a clear space of at least eighteen (18) inches between the shaft and the cofferdam for placing the forms.

Roof of Working Chamber

The thickness of the roof of the working chamber for pneumatic caissons shall conform to Table 43*a*.

For a width of pier exceeding 25 feet, it is preferable to use a centre longitudinal cutting edge to give support to the roof; and for widths over 30 feet, it is absolutely necessary to do so. For widths in excess of 25 feet, the strength of the roof should be tested; but it is better not to employ more than five courses of timber, in order to avoid an excessive settlement of the pier due to the compression of the wood in the roof. A material increase in strength and effectiveness will be gained by depositing a five-foot layer of concrete on top of the timber roof and letting this set for at least five days before starting to sink the caisson.

TABLE 43*a*

Thickness of Roof of Working Chamber for Pneumatic Caissons

Width of Base	Height of Base Cutting Edge to Shaft	Courses of 12" x 12" Timbers in Roof of Working Chamber
15′ or under	40′ or less..............	1 transverse, 1 longitudinal.
"	Over 40′................	2 " 1 "
15′ to 20′	Under 30′..............	1 " 1 "
"	30′ to 60′..............	2 " 1 "
"	Over 60′................	3 " 2 "
20′ to 25′	Under 20′..............	1 " 1 "
"	20′ to 40′..............	2 " 1 "
"	Over 40′................	3 " 2 "
25′ to 30′	Under 30′..............	2 " 1 "
"	Over 30′................	3 " 2 "

Bracing for Cribs and Caissons

In the caissons for open-dredging piers no special bracing is required; however, in those for pneumatic piers it is necessary to support the side walls at intervals of not more than fifteen (15) feet. This bracing shall consist of struts near the top and bottom of the working chamber framed into vertical 12″ × 12″ timbers bearing against the horizontal 12″ × 12″ wall timbers, as shown in Fig. 41*b*. A one and a half (1½) inch rod shall be placed on each side of the bottom strut, and shall extend through the walls of the caisson. The bottom struts and the tie rods shall be placed a sufficient height above the cutting edge to permit excavating underneath them.

Bracing for cribs of open-dredging piers shall be provided by extending every third horizontal well-timber both longitudinally and transversely to the outer walls and connecting it thereto by a half and half dap and a drift bolt. In pneumatic piers the walls of the cribs shall be braced at points not more than fifteen (15) feet apart in both directions, such bracing to consist of 12″ × 12″ timbers in every third or fourth wall-course. The bracing in the two directions shall be placed in adjacent courses so that the timbers can be bolted together where they cross each other. They shall be framed into the side-walls with a half and half dap and shall be drift-bolted to them. When the cribs are pumped out to any considerable depth, vertical waling timbers, 12″ × 12″, must be used between the ends of the bracing struts and the wall timbers. The timbers should be tested both in bending and bearing for the maximum head of water that is likely to come on them, so as to prevent overstress. At 1,200 pounds per square inch in bending, 12″ × 12″ timbers can resist a head of 16 feet of water on a 15-foot span. The bearing of the bracing struts on the vertical waling pieces generally needs the most careful attention. The bracing for cribs of pile piers should be similar to that for cribs requiring to be pumped out, and care must be taken to see that the piles are so arranged as not to interfere with the said bracing.

Cutting Edges

Where it is certain that logs or large boulders will not be encountered in sinking a pier, timber cutting edges of hard wood will suffice; but where there is the least doubt, steel cutting edges should be used. The latter should be formed of 6″ × 6″ or 8″ × 8″ angles of the heaviest sections and ¾″ or ⅞″ side-plates extending up the vertical sides of the chamber and thoroughly bolted thereto.

Bolts, Spikes, etc.

Timbers shall be fastened together every three (3) feet with ⅞″ round drift-bolts, two (2) inches shorter than the total thickness of the timbers fastened together, and driven into ¾″ round holes.

For the inner sheathing of pneumatic caissons, ⅜″ × ⅜″ × 8″ boat spikes shall be used, two (2) per square foot of sheathing. For 2″ sheathing on wells of open-dredging piers and on the outside of all piers, ⅜″ × ⅜″ × 6″ spikes are required; and for 3″ sheathing, ⅜″ × ⅜″ × 8″ spikes.

All adjustable bolts shall be ⅞″ round, and shall have 3″ diameter washers wherever they bear on timber.

Caulking

For pneumatic caissons all cracks inside of the working chamber are to be caulked with two (2) threads of oakum thoroughly driven to place. The heads of all spikes are to be wrapped with oakum before driving, and bolts are to be similarly wrapped under the washers.

For cribs all cracks between the horizontal timbers and also all cracks in the vertical sheathing are to be caulked with a single thread of oakum. It is not necessary in all cases for the caulking to begin at the cutting edge, but it should start at least ten feet below the probable surface of concrete inside when the crib is pumped out, and preferably below the ground surface.

Piles in Base

Piles shall be spaced not less than three (3) feet from centre to centre in both directions.

The elevation of cut-off of wooden piles shall always be below low water. For heights of base less than ten (10) feet, the cut-off should be midway between top and bottom of base, but not less than two (2) feet below top of base. For depths of base greater than twelve (12) feet, the piles may come to within four (4) feet of top of base. As pile cut-offs are paid for by the lineal foot, there is always a small saving in making the elevation of cut-off as high as possible. In detailing it is advisable always to show the tip of a pile cut off square and not pointed, as that is the way it will be driven.

Elevation of Top of Base

For channel piers the top of base should be about two (2) feet below extreme low water, and for piers on land at such a distance below ground surface as to ensure that the said base will never be exposed to view. In order to give a greater clear width of channel between piers, the top of base will sometimes have to be placed at a much lower elevation than that just mentioned.

Cofferdams

Cofferdams for building the pier shafts in the dry shall be of similar construction to the cribs. The bracing must be so arranged that it can be readily removed as the concrete is poured. The cofferdam shall be attached to the crib in such a manner that it can be easily detached after the pier is completed, at the same time giving a water-tight joint. Scabs can be used for this purpose, provided they can be pried off without trouble because of the depth of water; otherwise, removable rods attached to the crib below and to the cofferdam above must be employed.

Pier Shafts

Preference shall be given to the oblong shaft with rounded ends, except where an ice-break is needed. Where it is necessary to economize, the same form of pier should be adopted, but a well should be left in the centre, thus forming in reality two shafts connected by side walls. Where still greater economy is demanded, two circular shafts connected by a central web must be resorted to. The cheapest construction of all consists of two separate shafts, either round or square, not connected in any manner; and it should be employed only in extreme cases.

Where double shafts either with or without webs are adopted, they shall be spaced such a distance apart as to give the smallest size under coping. For adjacent spans having the same distance from centre to centre of trusses, the centre of the pier shaft should lie in the central plane of truss.

Batters

The shafts of piers shall have a batter of one-half (½) inch to the foot on all sides, except as specified under "Ice Protection" or when the stability of the pier or its appearance requires it to be greater.

Webs of Double-Shaft Piers on Single Bases

The single web connecting the shafts of a dumb-bell pier shall have a thickness of not less than two (2) feet at the top under coping; and at the bottom this thickness shall be not less than one-third (⅓) of the diameter of the shaft at that elevation. The sides of the web shall have a batter equal to that of the shafts, beginning at such a point below the top as to give the proper thickness at the bottom.

The web shall be reinforced from the top of the pier to a point at least five (5) feet below high water and at least ten (10) feet below the bottom of the coping. The vertical reinforcement shall consist of ¾″ round bars, spaced eighteen (18) inches from centre to centre in both faces; and ¾″ round bars of a length equal to the distance from centre to centre of shafts shall be placed horizontally in both faces twelve (12) inches apart on centres.

Walls of Double-Shaft Piers on Single Bases

The walls of double-shaft piers shall not be less than eighteen (18) inches thick for their full height. These walls shall be reinforced vertically in both faces with ¾″ round bars spaced eighteen (18) inches on centres and projected into both the coping and the base. Horizontal bars ¾″ in diameter shall also be placed twelve (12) inches on centres in both faces of each wall and extended to the centre of each shaft.

Coping

The coping shall be not less than two (2) feet thick, and shall project six (6) inches or more beyond the shaft at the top. When there is a centre well in the shaft, the coping is to cover the opening; and it shall be reinforced in both directions with ¾″ round bars twelve (12) inches on centres placed two (2) inches from bottom of coping. If the slab carries any superimposed load, it must be figured and properly reinforced for such load. For double shafts with central web, the coping shall have a constant width between shaft centres, and shall be reinforced as above, the reinforcement being placed two (2) inches below the top of the coping. The reinforcement shall be so arranged that it will not interfere with the anchor bolts.

The top of the coping for ordinary piers is to be finished ¾″ low, in order to allow for grouting under the shoes when no grillages are used; but when the latter are needed, they shall be set to correct elevation, and the top of the pier shall be finished flush with them. In both cases the 2-foot thickness of coping shall be measured from the under side of coping to the underside of shoe.

The top of the coping for pivot piers is to be leveled off with neat Portland cement mortar, and the lower track set in same. It shall be made one and one-half (1½) or two (2) inches higher in the centre than at the edge, so that the water will drain toward the latter. A small gutter or depression in the top of the pier is to be made just inside of the lower track; and at the bottom of this depression drain-holes are to be put in, leading the water from the gutter down on the outside of the pier. These drain holes are to be at least two (2) inches in diameter; and their tops are to be protected with screens, so as to prevent choking. They shall be spaced not to exceed ten (10) feet between centres.

Grillages and Anchor Bolts

Grillages shall be used whenever the pressure on the masonry would become excessive, unless the shoe were spread unduly. They shall consist of rolled or built-up beams, generally placed parallel to the length of the pier and arranged so as to give a clear space of at least four (4) inches between the adjacent flanges and so as to permit the placing of the anchor bolts. For grillages of moderate length the beams shall be connected at their ends by diaphragms, each consisting of a short piece of rolled channel and two angles turned out; for greater lengths, intermediate diaphragms shall be added to tie the beams together during fabrication. The top of the grillage shall be planed to receive the shoe.

The grillage shall be assumed to distribute the load from the shoe over an area equal to the number of beams multiplied by the distance from centre to centre multiplied by the length. It shall be proportioned for the moment occurring at the edge of the shoe due to the reaction of the pier on the cantilevered extension.

Anchor bolts shall be proportioned for the forces acting on them. They shall be fox-bolts when no calculable stress comes upon them, and regular anchor bolts with upset ends when figured for actual uplift. Fox-bolts shall be from ¾″ to 1½″ in diameter and from 12″ to 24″ long, the smaller sizes being adopted for plate girder and I-beam spans, and the larger ones for truss spans. Anchor bolts proper must be made of sufficient section and have ample embedment to resist, with a good margin of safety, any stresses that may come upon them. They shall, preferably, be set into the masonry at the time the concrete is poured, or if necessary, be grouted into drilled holes when the superstructure is being erected. In the former case gas-pipe sleeves two (2) feet long and of a diameter equal to the diameter of the bolt plus two (2) inches shall be placed in the top of the pier around the bolt so as to permit of adjustment when the shoes are set.

Ice Protection

Masonry piers or concrete piers faced with hard, durable stone do not require any additional protection from ice abrasion. Concrete piers, however, in streams subject to heavy ice and swift current, shall be protected by steel plates partially or wholly encasing the pier shaft and extending from the top of the base to a sufficient height to afford ample protection to the concrete. For shafts of moderate height the plates shall extend up to the underside of the coping. Where it is necessary to break up the ice, the up-stream end of the shaft shall be extended to form an inclined edge between the top of the base and some point above the highest possible ice level.

The metal for the ice protection shall be ⅜″ thick. Horizontal splices of 6″ × 3½″ × ⅜″ angles with the 3½″ outstanding leg on the lower side shall be spaced about five (5) feet apart on the inside and riveted to the main plates with one line of rivets on each side of the joint.

Where both ends of the pier are rounded, the plates shall be spliced with 12″ × ⅜″ vertical splice-plates placed on the inside at the point where the straight sides meet the rounded ends. Two lines of rivets shall be used on each side of the joint. When necessary, intermediate vertical splices of the same design shall be employed. The upstream edges of the plates covering an ice-break shall be spliced with an 8″ × 8″ × ¾″ angle on the outside and an 8″ × 8″ × ½″ angle on the inside, with two lines of rivets through each leg. If the bend in the angles is excessive, a 14″ × ¾″ plate on the outside and a 13″ × ½″ plate on the inside can be substituted for them. Where the faces of the nose meet the sides of the shaft, a 12″ × ¾″ plate is to be placed on the outside and a 12″ × ⅜″ plate on the inside to form the splice. The rivets shall be ⅞″ diameter spaced three (3) inches centres. The steel plates shall be anchored to the concrete with button-head bolts ⅞″ in diameter, threaded full length and having two nuts per bolt. They shall be spaced eighteen (18) inches centres in both directions.

Concrete Proportions

Concrete deposited beneath the water, as well as that placed in the coping, shall be mixed in the proportion of one (1) part of cement to two (2) parts of sand and three (3) parts of broken stone that will pass through a ¾-inch iron ring. Concrete in the working chamber of pneumatic caissons shall be mixed in the proportion of 1 : 2 : 4 with broken stone that will pass through a 1-inch iron ring. All other concrete shall be mixed in the proportion of 1 : 3 : 5 with broken stone that will pass through a 2½-inch iron ring. All concrete deposited beneath the water shall be placed with a bottom-dump bucket or through a water-tight trémie.

HIGHWAY CYLINDER PIERS

The minimum thickness of metal and the number of piles per cylinder for highway cylinder piers shall be as given in Table 43*b*. The piles should project up into the cylinders at least ten (10) feet.

Splices and Rivets

All splices are to be butt-splices with the splice-plates on the inside of the cylinder. Splice-plates shall be six (6) inches wide and of the same thickness as the cylinder metal. The horizontal splices shall be spaced about five (5) feet apart. Not more than two (2) vertical splices shall be put in any one ring. When there is only one vertical splice, the splices in adjacent rings shall be placed 180 degrees apart; and when there are two vertical splices, those in the same ring are to be spaced 180 degrees apart, and those in the adjacent rings 90 degrees apart. The rivets shall be $\frac{3}{4}$ inch diameter for metal $\frac{3}{8}$ inch thick and under, and $\frac{7}{8}$ inch diameter for metal over $\frac{3}{8}$ inch thick. Rivets shall be spaced three (3) inches centres.

TABLE 43*b*

HIGHWAY CYLINDER PIERS WITH PILES
(Thickness of Metal and Number of Piles)

Diameter of Cylinder	Minimum Thickness of Metal	Piles per Cylinder
Inches	Inches	
18	$\frac{5}{16}$	1
24	$\frac{5}{16}$	1
30	$\frac{5}{16}$	1
36	$\frac{5}{16}$	2
42	$\frac{5}{16}$	3
48	$\frac{3}{8}$	4
54	$\frac{3}{8}$	5
60	$\frac{3}{8}$	6
66	$\frac{3}{8}$	7
72	$\frac{3}{8}$	8 preferred 9 possible
78	$\frac{7}{16}$	11
84	$\frac{7}{16}$	13

Bracing between Cylinders

The bracing between cylinders shall be either a solid web or cross-struts with intersecting diagonal rods. The bracing shall extend from about two (2) feet above low water to the top of the pier. The diagonal rods in the open bracing shall be inclined at an angle of about forty-five (45) degrees, which inclination will determine the number of panels to be used. The rods shall be upset and provided with turnbuckles. The details of the two systems of bracing just described are illustrate in Figs. 43*a* and 43*b*; while Table 43*c* indicates the sizes of struts and diagonals to be adopted for various spans.

Highway Cylinder Piers without Piles

The specifications for cylinder piers with piles, except those for the piles themselves, are applicable to cylinder piers without piles. The latter type of pier can be sunk either by the pneumatic process or by open-dredging.

TABLE 43c

OPEN BRACING FOR CYLINDER PIERS

Length of Span	Size of Diag. Rods	Size of Pins	STRUTS FOR CLEAR DISTANCE BETWEEN CYLINDERS AS SHOWN			
			14′ or Less	14′ to 16′	16′ to 18′	18′ to 20′
20′	⅞″	1½″	2 [s 4″x 6¼ #	2 [s 4″x 6¼ #	2 [s 5″x 9 #	2 [s 5″x 9 #
40′	1⅛″	1¾″	2 [s 4″x 6¼ #	2 [s 4″x 6¼ #	2 [s 5″x 9 #	2 [s 5″x 9 #
60′	1⅜″	2 ″	2 [s 5″x 9 #	2 [s 5″x 9 #	2 [s 5″x 9 #	2 [s 6″x10½ #
80′	1½″	2½″	2 [s 5″x 9 #	2 [s 5″x 9 #	2 [s 6″x10½ #	2 [s 6″x10½ #
100′	1¾″	2¾″	2 [s 6″x10½ #	2 [s 6″x10½ #	2 [s 6″x10½ #	2 [s 6″x10½ #
125′	1⅞″	3 ″	2 [s 6″x10½ #	2 [s 6″x10½ #	2 [s 7″x12¼ #	2 [s 7″x12¼ #
150′	2 ″	3¼″	2 [s 7″x12¼ #	2 [s 7″x12¼ #	2 [s 7″x14¾ #	2 [s 7″x14¾ #

Pneumatic Cylinder Piers

Pneumatic cylinder piers shall be formed of two shells, an inner one and an outer one. The outer shell shall be of a size required for the pier base, while the inner one, forming the well to the air chamber, shall be three (3) feet in diameter, except in small piers, where this dimension may be reduced to two feet six inches (2′ 6″). The working chamber shall be formed by flaring out the inner shell for a height of about eight (8) feet at the bottom until it meets the outer shell at the lower edge, where a cutting edge, similar to that for a timber pier, is effected. The inner and the outer shells are to be braced together by open angle-trussing, preferably in six radial planes. This bracing shall extend up about five (5) feet above the flared portion of the inner shell, beyond which elevation the construction is to be similar to that of the cylinder piers with piles.

The plates for the lower thirteen (13) feet of the pier shall be $\frac{7}{16}$ inch thick; and the details of the cutting edge, bracing, and splices shall be as shown in Fig. 41c.

Highway Cylinder Piers in General

It may frequently occur that the size of cylinder required at the bottom is larger than that required at the top to take the span bearings, in which case the top may be reduced to the size required. The following points, however, should be kept in mind in doing so. The bottom 20 feet should be kept plumb so that the cylinder can be sunk with ease and accuracy. The reduction in size should be effected in one of two ways: (*a*) by telescoping the sections and putting in filling rings; (*b*) by making one five-foot section a frustum of a cone and thus concentrating all the reduction into that length. The cylinders can be uniformly battered for most of their length; but the shopwork would be more difficult than by either of the above methods. In pile piers the reduction in size must be at such a height as to allow the driving of the piles inside the cylinder after it is sunk; because, with the close spacing of piles necessary in this class of piers, it is not safe to depend on being able to drive the piles with sufficient accuracy to allow of the cylinders being sunk over them.

For the larger-sized cylinders it is not always necessary to carry the specified thickness of metal to the top. Thus above ordinary water surface it may be reduced to $^5/_{16}$ inch; and also in pneumatic piers, which will have concrete between the outer shell and the shaft during sinking, the metal above the ground surface may be made $^5/_{16}$ inch thick.

TIMBER PILE PIERS

Timber pile piers shall be proportioned to carry the maximum superimposed loads without exceeding the allowable load on piles. The piles shall be arranged so that the load will be distributed uniformly over them. Where pile piers are to be replaced later by more permanent construction, the piles shall be so located that they can be incorporated into the final design, if such an arrangement be practicable. Substantial bracing bolted to the piles shall be employed; and in streams with any appreciable current and carrying either ice or drift, the up-stream end shall be battered and brought to a point so as to form a nose or breakwater; and the entire group of piles shall be enclosed by four (4) inch oak sheathing, laid close together and filled inside with stone. In cases where the ice is very destructive, the face of the breakwater shall be covered with metal protection.

CHAPTER XLIV

SHORE PROTECTION AND MATTRESS WORK

RIVERS with shifting channels and friable banks often render it necessary to resort to shore protection in the vicinity of a bridge in order to prevent the current from cutting in behind the structure. Also, the possibility of an excessive depth of scour around the channel piers during floods makes it desirable in some cases to protect each foundation by a willow mattress placed on the bed of the stream and surrounding the pier. As the results of any such local protection work extend some distance from the locality of the improvement, it is incumbent upon the engineer in making the plans to consider its probable effect on the regimen of the river both above and below the bridge site. A change in the velocity or direction of the current in one section causes a readjustment of the regimen that extends to contiguous sections. For this reason it would be preferable and conducive to greater permanency of the protection, if local improvements were made to conform with and become a part of a general scheme for the entire improvement and control of the river. In seeking to protect the bridge by controlling the current, it must be recognized that the natural channel should consist of a series of advantageous curves. To attempt the elimination of any such curves is unwise, as that would produce an increased slope and a correspondingly higher velocity with further scouring until a new bend is formed. It should also be borne in mind that any obstruction causing an abrupt change in the direction of the current involves a loss of its kinetic energy which is expended in a whirl that causes scour and produces disturbances in the regimen. The effort of the engineer should be to control the stream by making the concave margin of the bend a true director of the current and to avoid diverting an excess of flow to the opposite side, thereby unnecessarily deepening that margin and causing a rebound toward the initial shore line.

For the purpose of controlling the current, it is usually better to adopt shore revetment rather than groynes, or spur dykes. While the latter, when properly designed and constructed, have proved effectual, experience has shown that their results are uncertain unless the works are planned by a river specialist having full and reliable data at hand covering the elements of the physics of the stream in question. Spur dykes involve abrupt changes in velocity and direction of current which produce results difficult to foresee. For a detailed discussion of methods of river control, the reader should refer to the excellent article by the eminent hydraulic engineer, S. Waters Fox, in Vol. LIV of the *Trans.* Am. Soc. C. E., or to Professor Van Ornum's standard work entitled "The Regulation of Rivers."

Besides the direct action of the current, the other contributing factors to bank erosion are surface flooding from a heavy downpour of rain, overpour from local pondage, seepage or subsurface flow from some nearby storage, and wave action. The extent to which erosion, or bank caving, results from any of these causes obviously depends upon the character of the material composing the said bank, the homogeneity thereof, and the rapidity with which the water is brought into contact with it.

Banks subject to caving or sloughing by reason of ground water, seepage, or subsurface flow can be most effectually protected by thorough drainage. If the seepage comes from visible sources, this is readily accomplished; and the methods to be employed will be obvious on inspection of the situation. One of the most troublesome conditions is where the source of the ground flow is not visible and where the water finds its way through a substratum of quicksand in the bank. In this case drainage will prove helpful. When the substratum is not too deep, this may sometimes be accomplished by driving sheet piling through the water-bearing stratum on a line well inshore from the bank and providing a means of getting rid of the water which collects against the dam thus formed, either by drain-pipes or, if absolutely necessary, by pumping.

When the erosion is due to wave action, the bank should be graded to a slope at least as flat as the natural slope of the saturated material of which it is composed. A slope of one to three will usually meet all conditions. After grading, the bank should be either covered with willow mattress or paved with stones set, preferably, on edge and in such position that the pieces will incline inshore from a vertical. The laying of the stone should, therefore, be commenced at the top of the bank and progress downward. The voids should be filled with spawls or crushed stone. In extended reaches of the bank subject to high winds and persistent wave action of some violence, it sometimes becomes necessary to break up the lateral transmission of waves by means of short spur dykes.

When the erosion is due to the action of the current, a willow mattress covering the subaqueous portion of the bank and extending from a little above low water-mark well out over the river bed is an effective device. This mattress should be continuous along the concave shore from the beginning of the bend to the point below where the stream axis is deflected to the opposite bank. Above the water-line, the bank after being graded to a proper slope for stability should be riprapped as previously described. The advantage of the willow mattress is its flexibility, permitting of ready adjustment to irregularities of the stream bed. By extending this mattress sufficiently beyond the toe of the slope and loading it with large stones, it will follow down any scour in the river bed and thus afford continual protection to the subaqueous portion of the bank. For all cases where it is desired to prevent scouring of the river bed or the subaqueous portion of the bank, the willow mattress is perhaps the surest and most economical means of obtaining the desired result.

In case of channel piers threatened by excessive scour, a mattress can be placed on the river bed covering the pier location and weighted into position with heavy stones. Then the caisson can be sunk to the mattress, the interfering stones removed, and a hole cut through the brush of the size and shape of the caisson, so as to permit the latter to pass through without disturbing the mattress. This is the ordinary method of procedure, but the author prefers to weave a hole in the mattress for the caisson to pass through, leaving ample margin all around for error in placing. After the mattress is down and anchored, the space between it and the caisson should be filled with fascines or "babies" consisting of long bunches of willows well wired together, each containing one or two large stones for sinking. These fascines should not only fill the space between crib and mattress, but should also extend above it about a foot to provide for crushing and settling, after which a layer of large stones should be placed above them to serve as rip-rap. The author's experience has shown that in most cases a margin of three feet all around will make the hole large enough, provided that due precaution be taken in locating the mattress. If much difficulty be anticipated from swift current, the margin should be increased; but by putting in four diagonal anchorages (either pile or mushroom), one from each corner, and attaching the mattress thereto with wire rope in such a manner that it may be shifted in any horizontal direction, its location can be made as accurate as desired.

Two standard types of mattresses have been evolved—the basket-woven type and the solid-fascine type. The woven mattress is usually composed of willow poles of small diameter, interlaced and woven together like a mat, strengthened by a system of galvanized wire rope running lengthwise and crosswise of the mattress, and sewn to the selvage and intermediate ropes with strong wire. The solid-fascine type, which of late years on the lower Mississippi has almost displaced the woven mattress, consists of bundles, or fascines, of willows laid parallel to each other and bound closely together by means of wire strands which run continuously through the mattress.

Specifications covering mattress construction will be found in Chapter LXXIX.

In addition to the use of mattresses for shore protection, it is sometimes found desirable to modify the shore line in places so as to provide a more advantageous curve, in which case a "training" dyke is built on the desired alignment, after a mattress has been placed on the river bed to prevent scouring and the consequent undermining of the dyke. The usual construction of these training dykes consists of two or more rows of piles driven about 8 ft. apart, with piles staggered, forming equilateral triangles having sides about 10 ft. long. These rows are then braced and tied together with wales and struts so as to form a unit, and the space between the rows of piling is filled with rock up to low water, while the upper part is made to act as a screen either by wattling or by nailing small vertical poles to the waling. The cross-dykes are treated in a similar manner. The effect is to cause accretions behind the dyke, thereby establishing a new shore line, and at the same time giving a direction to the current that will benefit the channel. Single spur-dykes have been tried for the purpose of deflecting the current and thus preventing the encroachment of the stream. There are two serious defects to this system: First, immense quantities of driftwood find lodgment against the dyke in unevenly distributed masses. This often results in the destruction of the dyke by overturning or by crushing; and at the same time the evenness of the deposits behind the dyke is interfered with. The second trouble is that a pot-hole forms at the stream end of the dyke and prevents the formation or maintenance of deposits there. There is more or less of an eddy continually occurring at the end of the structure; and, as decay sets in, it yields to the force of the current; and when once the end of the dyke is destroyed the rest succumbs quickly.

Another type of cross-dyke used for protecting the toe of a bank subject to scour is made of sink-fascines. These are like the "babies" previously described, but larger, being formed of bundles of willow poles, each bundle being four or five feet in diameter and containing enough rocks in the interior to provide the weight required to hold them on the river bed. These sink-fascines are placed on the bottom parallel with the shore line, and are further anchored by clusters of piles driven at intervals. The top of this pavement of fascines is made to conform approximately with the desired slope of bank, so that as the sand and silt are deposited around them a new slope is gradually formed. An arrangement of this kind was advocated in a report by the author upon how best to protect the south bank of the Missouri River above the Hannibal Bridge at Kansas City.

Another type of bank dyke is that invented by a Mr. David Neale. It consists of hollow or cellular fascines anchored to the shore and floated into place, where they collect silt and settle to the bed of the river. The sinking may be hastened by placing bags of sand inside the fascines. These fascines in one or two layers form the foundation upon which to build the mud cells. These are constructed after the manner of a log house with many rooms from 5 feet to 8 feet square. They are built in and over the upper slope of the bank, which has previously been graded to the proper angle. These mud cells are stepped back on the water front to conform roughly to the slope of the bank, and the top is covered with a layer of brush. Rip-rap is used at the top of the bank to make it more permanent. These dykes are subject to scour at the end.

They have been used successfully on both the Missouri and the Arkansas rivers; but the author much prefers the old-fashioned type.

Concrete revetment has been adopted in several cases on the Missouri River. This consists of paving the bank above standard low water with strips or slabs of reinforced concrete about eight feet wide, and of placing at water-line and extending outward over the willow mattress a system of flexible concrete block protection formed of blocks about twenty-four inches square, linked together and to the solid concrete paving. Concrete revetment has been used near the mouth of the Kaw River to protect the levees. It is laid directly on the earth embankment in large slabs. A number of cracks have developed in it, but thus far it is holding its own very well.

No general rule can be laid down for determining which of these various systems of shore protection is applicable or best for any particular case without making a special study of the actual conditions at the site.

The possibility of the necessity for river protection work and the expense of its construction should always be given due consideration when estimating on the cost of any important bridge.

Mattresses are generally kept in place by anchoring them with wire ropes to single piles, pile clusters, dead-men on the bank, or large mushrooms anchors in the stream. In the middle eighties there was invented by John W. Nier, Esq., C. E., a method of anchoring mattresses that was used in a few places with great success, and then, for some unknown reason, was abandoned, so that today its scheme of attachment may be considered a lost art. It consisted in driving at regular intervals through the mattress, by means of a "harpoon" actuated by a water-jet, thin circular cast-iron mushroom anchors, each eight inches in diameter and fastened at the centre to a piece of telegraph wire, the penetration into the soil being about ten feet. The point of the harpoon was easily worked through the brush, after which it went down to the full depth in a few seconds. By shutting off the water and giving simultaneously to the apparatus a sudden jerk, the anchor was released in such a manner as to take a horizontal position. By stressing the wire and attaching it to the skeleton ropes of the mattress, an excellent anchorage was obtained. The first practical trial of this scheme on actual construction was made by the author about 1890 during the construction of the Pacific Short Line Bridge over the Missouri River at Sioux City, Iowa. The river bank above the bridge on the Nebraska side was caving, and hence had to be protected. The author, having great faith in the efficacy of his friend's invention, gave Mr. Nier a contract for doing the protection work; and the result justified his action, for the caving was checked and the mattress did its duty well until the willows rotted some years later. Meanwhile, however, the channel of the river had shifted to the Iowa side, where it has remained till this day, rendering unnecessary the further protection of the Nebraska bank.

When the East Omaha Bridge across the Missouri River was being built a few years later, the stream gave indications that the channel would shift from the Iowa to the Nebraska shore, and that, in consequence, the East Omaha Land Company's property would be cut away. To prevent this the author designed and built a long, trailing, wattled-pile dyke well out into the stream with wattled-pile cross-dykes at intervals, the piles of the trailing dyke being driven through a willow mattress that was anchored by the Nier method. The protection started on a tangent some distance above the bridge site at the beginning of a convex curve in the shore line, deflected gradually to the crossing, where it passed close in front of the last pier of the temporary bridge, and then continued for some distance down stream.

White oak piles about fifty feet long with 8″ × 10″ oak timbers for caps and 6″ × 8″ yellow pine timbers for bracing were used. The piles were driven five (5) feet from centre to centre in two rows six feet apart, and the rear row was wattled by weaving slender poles in and out between them, and pushing the poles down to the mud, thus forming a fence, through which the water in passing was forced to drop a portion of its burden of sand. The fifty-foot-wide mattress, most of which was outside of the dyke, prevented any scouring out of the piles. At the upper end of the dyke the construction was built specially strong and attached by heavy wire rope to a dead-man set well back into the bank. Not only did this dyke answer perfectly the purpose of protecting the river bank, but also, incidentally, it formed a large accretion, thus adding materially to the area of the saleable lands of the East Omaha Land Company.

At the time of the completion of the permanent bridge some twelve years later the dyke was still in good shape, although injured a little in places by the ice, for the timber and piles had not decayed. Whether any trace of the old work remains today after the passage of more than two decades since its building, the author was curious to ascertain, consequently he wrote to A. S. Baldwin, Esq., C. E., Chief Engineer of the Illinois Central Railway Company, which company is the present owner of the East Omaha Bridge, and asked him to tell him the condition of the old construction. Mr. Baldwin, not having the information at hand, was so kind and courteous as to take the trouble to send an engineer to East Omaha with instructions to investigate and report upon the matter. The result was that on April 10, 1915, Mr. Baldwin wrote the author a letter, from which the following extract is made:

> "Referring to your letter of March 24th in regard to condition of the old pile dykes built near the East Omaha bridge to protect the shore line some twenty years ago.
>
> "We had no definite information in the office in regard to the condition of this dyke, and I arranged to have a representative go to Omaha and look over the work with a view of obtaining the information which you desire.
>
> "I attach hereto photographic view and sketches showing the type of construction of these dykes on both sides of the river near the bridge.

"The dyke on the Nebraska side is about 2200 feet long and has five cross-dykes. All portions of the structure are completely covered, with the exception of a section of the parallel dyke near the bridge, which is shown in the picture. This dyke is built of two rows of white oak piles with 8 x 10 inch caps and 6 x 8 inch braces of yellow pine. The material which is exposed above the job is in very good condition, and I presume the cross-dykes are in the same shape as those on the Iowa side, but it was impossible to determine, as they were completely submerged. The Government map which is attached indicates that the dyke has served the purpose for which it was intended, as there is very little difference between the shore line in 1890 and the present one. The bank at this point is filled to a height of 12 feet above the top of the parallel dyke."

Fig. 44*a* shows the only portion of the trailing dyke that has not been covered by a deposit of sand.

The author has made a practice of building his mattresses generally about twelve inches in thickness, although common practice permits of ten inches; and in special constructions, such as pier mats, he has made them sixteen and even eighteen inches thick. In order to prevent the anchor stones from being washed off, he lashes firmly to the skeleton ropes of the construction small logs running in two directions at right angles to each other so as to form cribs for holding the load. These serve to stiffen the mattress, but not enough to prevent its settling when undermined.

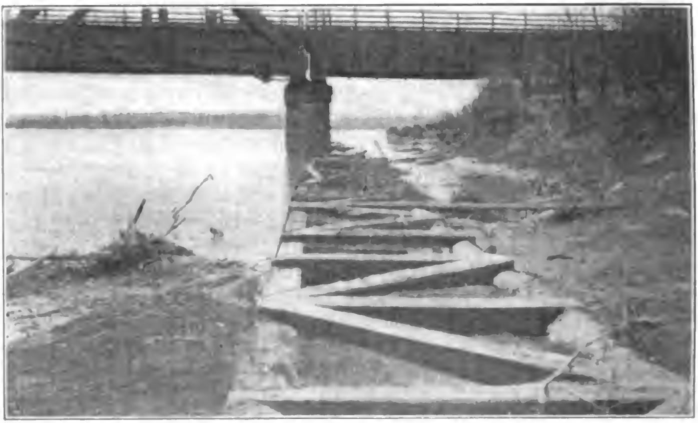


FIG. 44*a*. Shore Protection on the Nebraska Side of the Missouri River at East Omaha, Neb.

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